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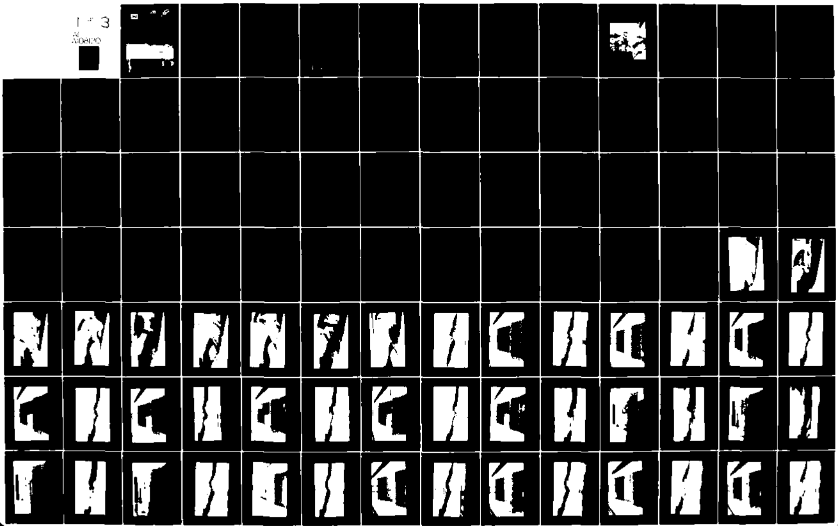

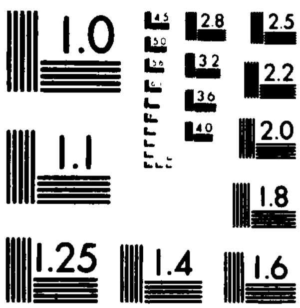

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TECHNICAL REPORT HL-81-11

BREAKWATER AND REVETMENT STABILITY STUDY, SAN JUAN NATIONAL HISTORIC SITE SAN JUAN, PUERTO RICO

Hydraulic Model Investigation

by

Dennis G. Markle

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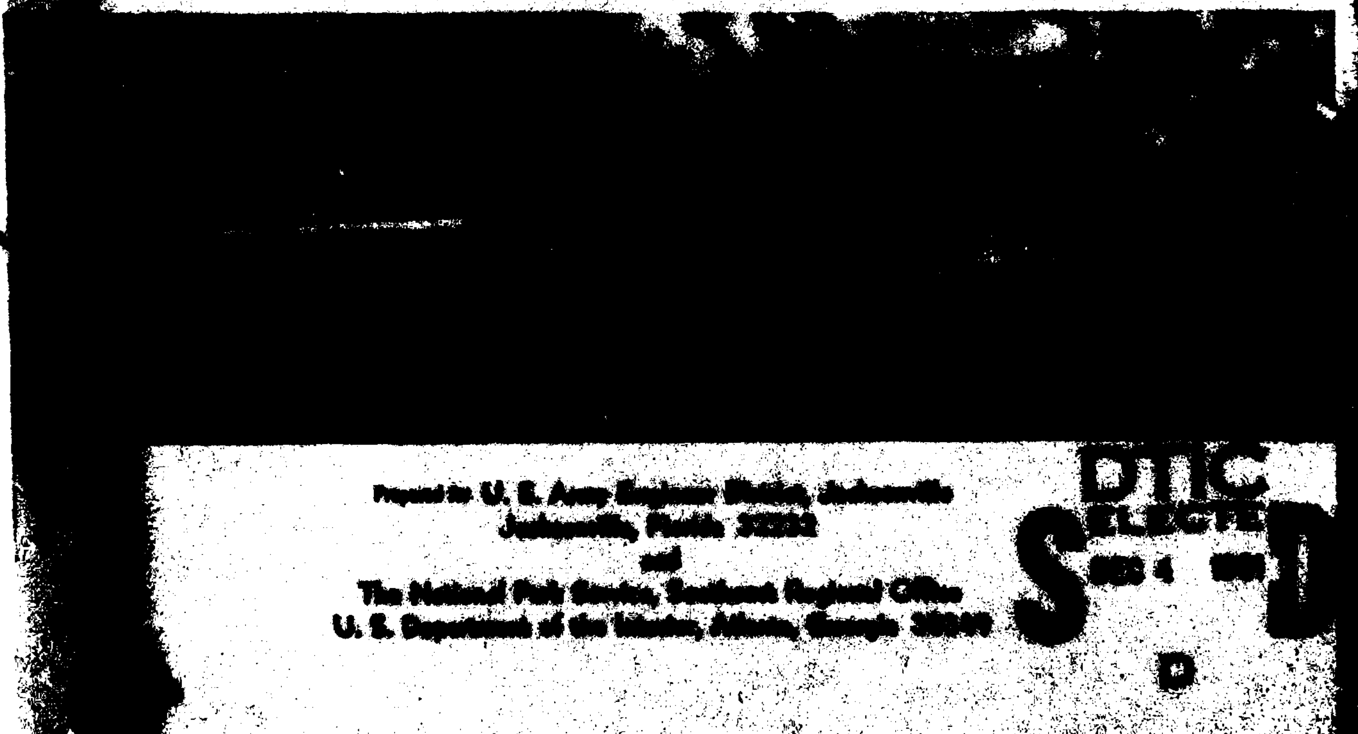
September 1981

Final Report

Approved For Public Release; Distribution Unlimited



Prepared for U. S. Army Engineer District, Jacksonville
Jacksonville, Florida 32232
and
The National Park Service, Southeast Regional Office
U. S. Department of the Interior, Atlanta, Georgia 30303

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A hydraulic model investigation was conducted using both two-dimensional and three-dimensional stability models at undistorted linear scales (model to prototype) of 1:38.5 and 1:50.5, respectively. The purposes of the stability tests were as follows:

a. Develop stable, economical and aesthetically pleasing designs for the offshore breakwater, north revetment, and west revetment to protect the

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20. ABSTRACT (Continued).

San Juan National Historic Site from storm conditions that would generate depth-limited breaking waves at still-water levels (swl's) of 0.0 and +1.9 ft msl.

b. With the protective structures in place, determine the runup produced on the north and west slopes when exposed to a range of wave heights with wave periods from 7 to 17 sec at swl's of 0.0 and/or +1.9.

c. Both with and without the offshore breakwater and north revetment in place, expose the construction trestle to a range of wave periods and wave heights at swl's of 0.0 and +1.9 to observe the impact of the combined incident and reflected waves on the trestle and support pilings. Test results indicated that a two-layer system of 28-ton armor stone, placed on a 1V-on-2H slope using random placement, would be an adequate design for the offshore breakwater. Both the north and west revetments were constructed with slopes of 1V on 3H and two-layer, random armor-stone placement. Five-ton armor stone was adequate on the protected north revetment, while 12-ton armor stone was needed for stability of the unprotected north revetment. Three-and-one-half-ton armor stone proved to be adequate for the unprotected west revetment design.

For wave periods ranging from 7 to 17 sec at swl's of 0.0 and +1.9 incident wave heights (measured at the -21.0 contour on the north slope) greater than 10.0 ft could create conditions that would be potentially damaging to the construction trestle. These conditions appeared to be more severe with only the trestle in place; however, they also occurred when the trestle, breakwater, and protected north revetment were on the north slope concurrently.

PREFACE

In October 1974 Congress authorized the Secretary of the Interior in cooperation with the Secretary of the Army to conduct studies to determine the cause and extent of damage to the historic structures of the San Juan National Historic Site. As part of these investigations, a sequence of model studies was agreed upon to provide data that would determine the most suitable plan for shore protection and restoration of the foundation walls along the shores of the historic site.

The model investigation reported herein was initially requested by the U. S. Army Engineer District, Jacksonville (SAJ), in a letter to the U. S. Army Engineer Waterways Experiment Station (WES) dated 17 September 1979. Funding authorization by SAJ was granted in SAJ Intra-Army Order No. 08-123-ENG-169-77, dated 29 July 1977 and amendments thereto.

Model tests of the breakwater and revetments stabilities were conducted at WES during the period September 1979 to September 1980 under the general direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, Dr. R. W. Whalin, Chief of the Wave Dynamics Division, and Mr. D. D. Davidson, Chief of the Wave Research Branch. Tests were conducted by Mr. D. G. Markle, Hydraulic Research Engineer, assisted by Messrs. V. L. Copeland, C. R. Herrington, and C. Lewis, Engineering Technicians. This report was prepared by Mr. Markle.

Liaison was maintained during the course of investigation by means of progress reports and telephone conversations.

Commanders and Directors of WES during the conduct of this study and the preparation and publication of this report were COL Nelson P. Conover, CE, and COL Tilford C. Creel, CE. Technical Director was Mr. F. R. Brown.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
inches	25.4	millimetres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square feet	0.09290304	square metres
tons (2000 lb, mass)	907.1847	kilograms

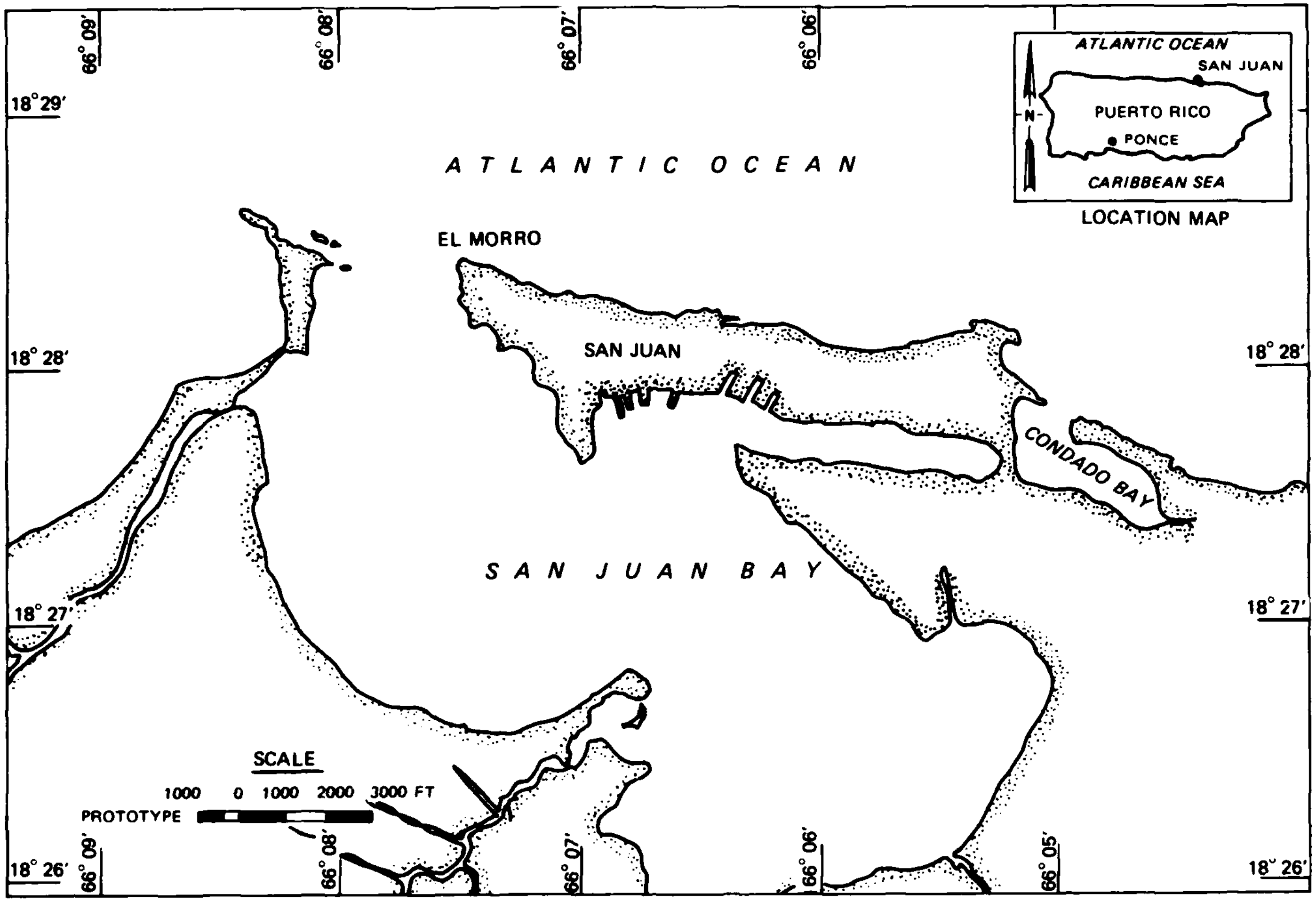

Figure 1. Project location map

BREAKWATER AND REVETMENT STABILITY STUDY

SAN JUAN NATIONAL HISTORIC SITE

SAN JUAN, PUERTO RICO

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. During the 16th and 17th centuries, a fortification complex was constructed on the northern coast of Puerto Rico (Figure 1). The complex served as a defense for the old city of San Juan and a base of operations for the Spanish influence in the Americas. To ensure the preservation of the fortifications, the San Juan National Historic Site was established by the Secretary of the Interior on 14 February 1949 and includes the fortifications of La Princesa, San Cristobal, Castillo de San Felipe del Morro (El Morro Castle), and numerous connecting walls and bastions.

The Problem

2. Years of direct wave attack on the cliffs surrounding the San Juan National Historic Site have resulted in extensive scour and erosion. Large caverns and overhanging rock ledges have been carved out of the cliffs and are threatening the structural integrity of the rock foundations and walls of historic fortifications. Figure 2 shows a typical example of the eroded conditions of the cliffs surrounding El Morro Castle.

Proposed Protective Structures

3. To protect the deteriorating foundation and walls of El Morro Castle from future storm waves, a combination of offshore breakwater and stone revetments was proposed and tested in a three-dimensional

Figure 2. Typical eroded condition of slopes surrounding El Morro Castle

(3-D) wave action model* to determine the optimum revetment locations and breakwater position and alignment. The final design of proposed protection would consist of an offshore breakwater and stone revetment on the northern, or open-ocean, side of El Morro Castle and a stone revetment on the western, or bay, side of El Morro Castle (Figure 3). The remaining walls and cliffs surrounding the historic fortifications will be protected with stone revetments on both the open-ocean and bay sides.

Purpose of the Model Study

4. At the request of the U. S. Army Engineer District, Jacksonville (SAJ), two-dimensional (2-D) and 3-D breakwater and revetment stability tests have been conducted by the U. S. Army Engineer Waterways Experiment Station (WES). The purposes of these stability tests were as follows:

a. 2-D stability tests (wave attack at a 90-deg angle to the structure).

(1) Develop stable, economical, and aesthetically pleasing designs for the trunk of the offshore breakwater, the north revetment, and the west revetment to protect the San Juan National Historic Site from storm conditions that would generate depth-limited breaking waves at still-water levels (swl's)** of 0.0 and +1.9 ft† mean sea level (msl).††

(2) With the offshore breakwater and north revetment in place, determine the runup produced on the north slope by a range of wave heights with wave periods from 7 to 17 sec at swl's of 0.0 and +1.9.

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* R. R. Bottin, Jr. 1979 (Sep). "San Juan National Historic Site, San Juan, Puerto Rico, Design for Prevention of Wave-Induced Erosion; Hydraulic Model Investigation," Technical Report HL-79-15, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

** For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix A).

† A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

†† All elevations (el) cited herein are in feet referred to mean sea level (msl).

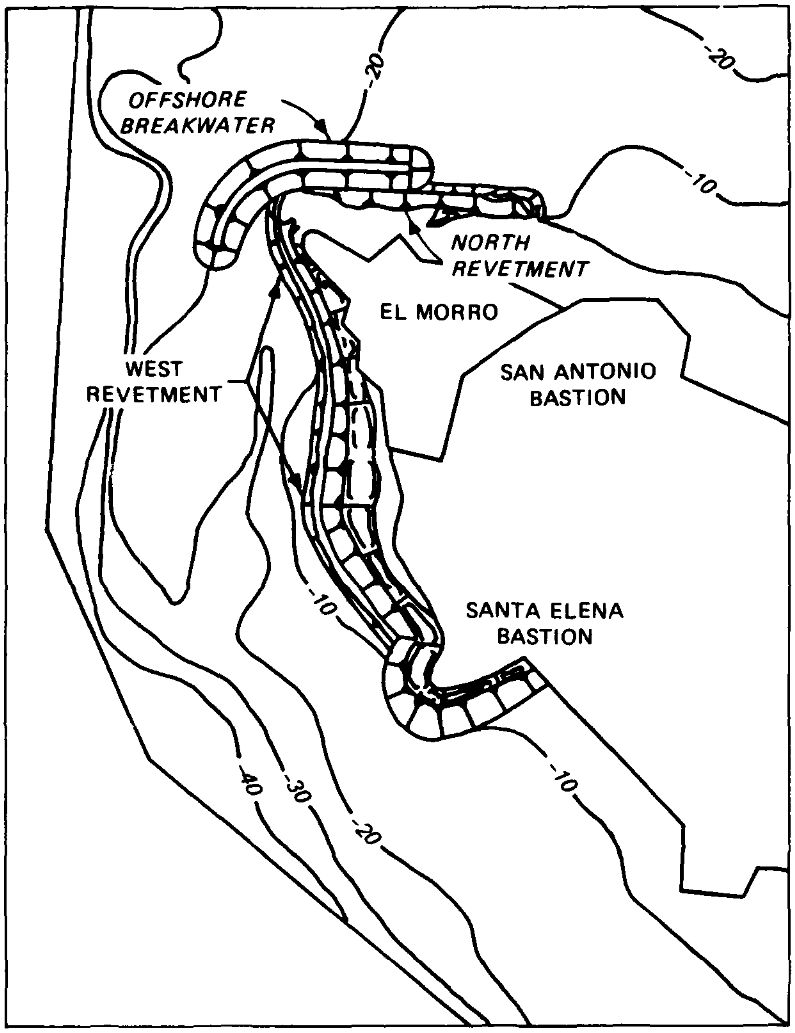


Figure 3. Proposed protective structures: offshore breakwater, north revetment, and west revetment

(3) With the unprotected west revetment in place, determine the runup produced on the west slope for a range of wave heights with wave periods from 7 to 17 sec at an swl of +1.9.

(4) Both with and without the offshore breakwater and north revetment in place, expose the construction trestle to a range of wave periods and wave heights at swl's of 0.0 and +1.9 to observe the actions of the waves on the trestle and its support pilings.

b. <u>3-D stability tests</u> (wave attack at angles other than 90 deg to the structure).

(1) Check the stability of the head and adjacent trunk of the offshore breakwater for the breaking wave conditions which could occur at swl's of 0.0 and +1.9 for incident wave directions of north, N30°W, and N72°W.

(2) If the armor-stone weight, found to be stable on the trunk of the breakwater during the 2-D tests, proves to be unstable on the breakwater head and adjacent trunk, optimize design of the breakwater head and adjacent trunk.

PART II: THE MODELS

Design of the Models

5. The 2-D and 3-D stability tests were conducted at geometrically undistorted linear scales (model to prototype) of 1:38.5 and 1:50.5, respectively. Scale selections were determined by the absolute size of the model breakwater and revetment sections necessary to preclude stability scale effects,* available model armor-stone weights, capabilities of available wave generators, and depths of water at the toes of the breakwater and revetment sections. Based on Froude's model law** and linear scales of 1:38.5 and 1:50.5, the following model to prototype relations were derived. Dimensions are in terms of length (L) and time (T).

Characteristics	Dimensions	Model-Prototype Scale Relations 1:38.5 Scale	1:50.5 Scale
Length	L	$L_a = 1:38.5$	1:50.5
Area	L^2	$A_a = L_r^2 = 1:1482.3$	1:2550.3
Volume	L^3	$V_a = L_a^3 = 1:57{,}066.6$	1:128,787.6
Time	T	$T_a = L_a^{1/2} = 1:6.2$	1:7.1

6. The specific weight of water used in the model was assumed to be 62.4 pcf; that of seawater is 64.0 pcf. Specific gravities of the model and prototype construction materials were identical. The difference in specific gravity of the model fresh water and the prototype seawater was accounted for by use of the following transference equation:

* R. Y. Hudson. 1975 (Jun). "Reliability of Rubble-Mound Breakwater Stability Models," Miscellaneous Paper H-75-5, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

** J. C. Stevens et al. 1942. "Hydraulic Models," Manual of Engineering Practice No. 25, American Society of Civil Engineers, New York.

$$\frac{(W_r)_m}{(W_r)_p} = \frac{(\gamma_r)_m}{(\gamma_r)_p} \left(\frac{L_m}{L_p}\right)^3 \left[\frac{(S_r)_p - 1}{(S_r)_m - 1}\right]^3$$

where

subscripts m, p = model and prototype quantities, respectively

W_r = weight of individual stone, lb

γ_r = specific weight of an individual stone, pcf

γ_w = specific weight of water, pcf

L_m/L_p = linear scale of the model

S_r = specific gravity of an individual stone relative to the water in which it was placed, i.e., $S_r = \gamma_r/\gamma_w$

Method of Constructing Test Sections

7. All model breakwater and revetment sections were constructed to reproduce, as closely as possible, the type of construction that can be achieved in the prototype. The core materials were dumped by bucket or shovel and leveled to grade. Hand trowels were used to compact the core materials in an effort to simulate the natural consolidation that occurs due to wave action during the construction period. The primary armor-stone or dolos layers, two armor units thick, were constructed by placement of the armor in a random manner. Random construction means that no conscious efforts were made to achieve a pattern or special placement technique. Photos 1-9 show the construction of a typical 2-D test section for the north slope stability tests. As shown in the photographs, building of the 2-D test sections is controlled by drawings on the sidewalls. In building the 3-D test section, the grade and slope of the subgrade material are controlled by templates, but by necessity the armor material is template-free and is controlled by an engineer's level. The 3-D test section was constructed on a metal baseplate to allow the structure to be repositioned for wave attack from various directions.

2-D Test Flume

Flume geometry and wave generator

8. All the 2-D stability tests were conducted in a 5-ft wide, -ft-deep, and approximately 124-ft-long concrete flume. The flume was equipped with a vertical displacement wave generator capable of producing monochromatic waves of various periods and heights. Figure 4 gives an elevation view of the test flume.

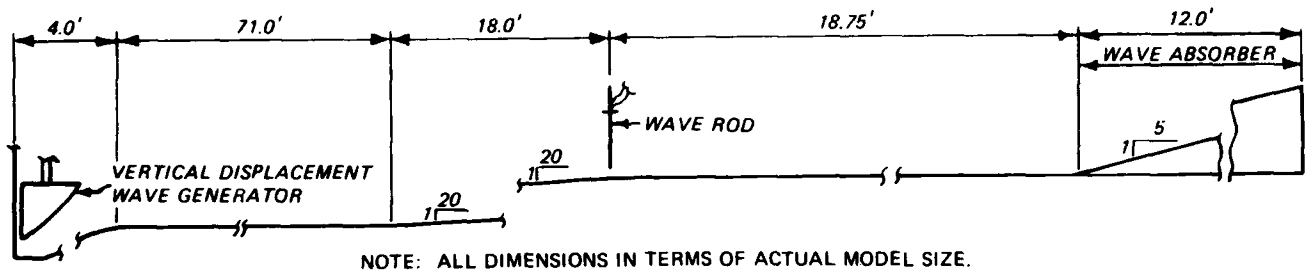
Test flume calibration

9. North slope. During calibration, water-surface elevations were measured by an electrical wave-height rod and recorded on chart paper by an electrically operated oscillograph. Since the seaward toes of the offshore breakwater and the unprotected north slope revetment were located at about the -21 and -18.2 ft contours, respectively, and since these contours were relatively close to the shoreline (where undesirable reflected wave energy can interfere with the incidence waves), wave heights for the flume calibration were measured at the -21 and -18.2 ft locations without the proposed structures and shoreward foreslopes in place. A wave absorber was installed in the landward end of the flume to reduce reflected wave energy. The top of the 1V-on-2H slope (Figure 4) represented the -21.0 ft contour in the prototype.

10. West slope. The same method of flume calibration was used as discussed in paragraph 9, but the wave rod was positioned at the point where the toe of the proposed west slope revetment met the existing bottom elevation (el -13.0), as shown in Figure 5.

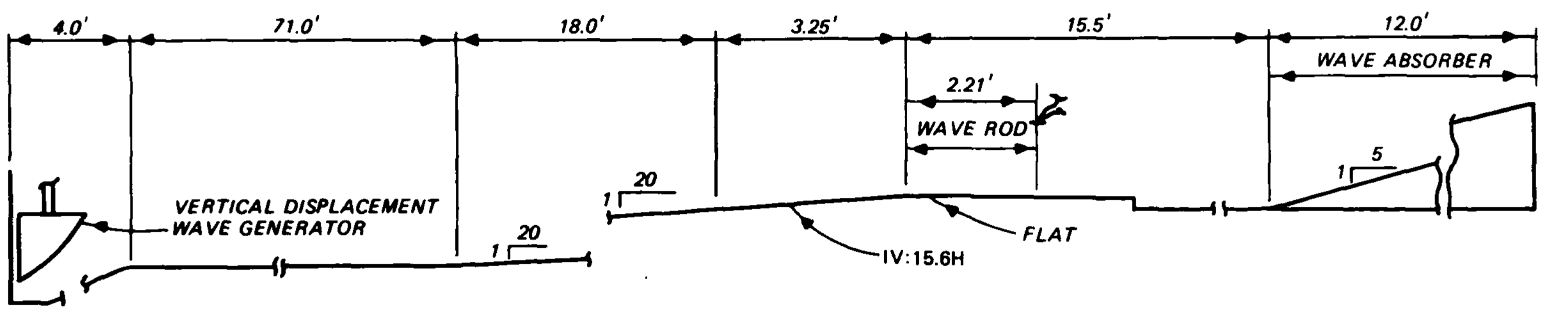
Modeling local bathymetry

11. The north and west slopes, as shown in Figure 6, were selected for testing the offshore breakwater and north revetment and the west revetment, respectively. The north slope, from the -21.0 ft contour to the +50.0 elevation on the castle wall, was modeled in the 2-D test flume (Figure 7) at the conclusion of the north slope flume calibration. After testing with the north slope, the north slope was removed and the west slope above the -21.0 ft contour was molded in the 2-D test flume (Figure 8).



NOTE: ALL DIMENSIONS IN TERMS OF ACTUAL MODEL SIZE.

Figure 4. Flume geometry and wave rod location for calibration of the 2-D test flume for stability tests on the north slope



NOTE: ALL DIMENSIONS IN TERMS OF ACTUAL MODEL SIZE.

Figure 5. Flume geometry and wave rod location for calibration of the 2-D test flume for stability tests on the west slope

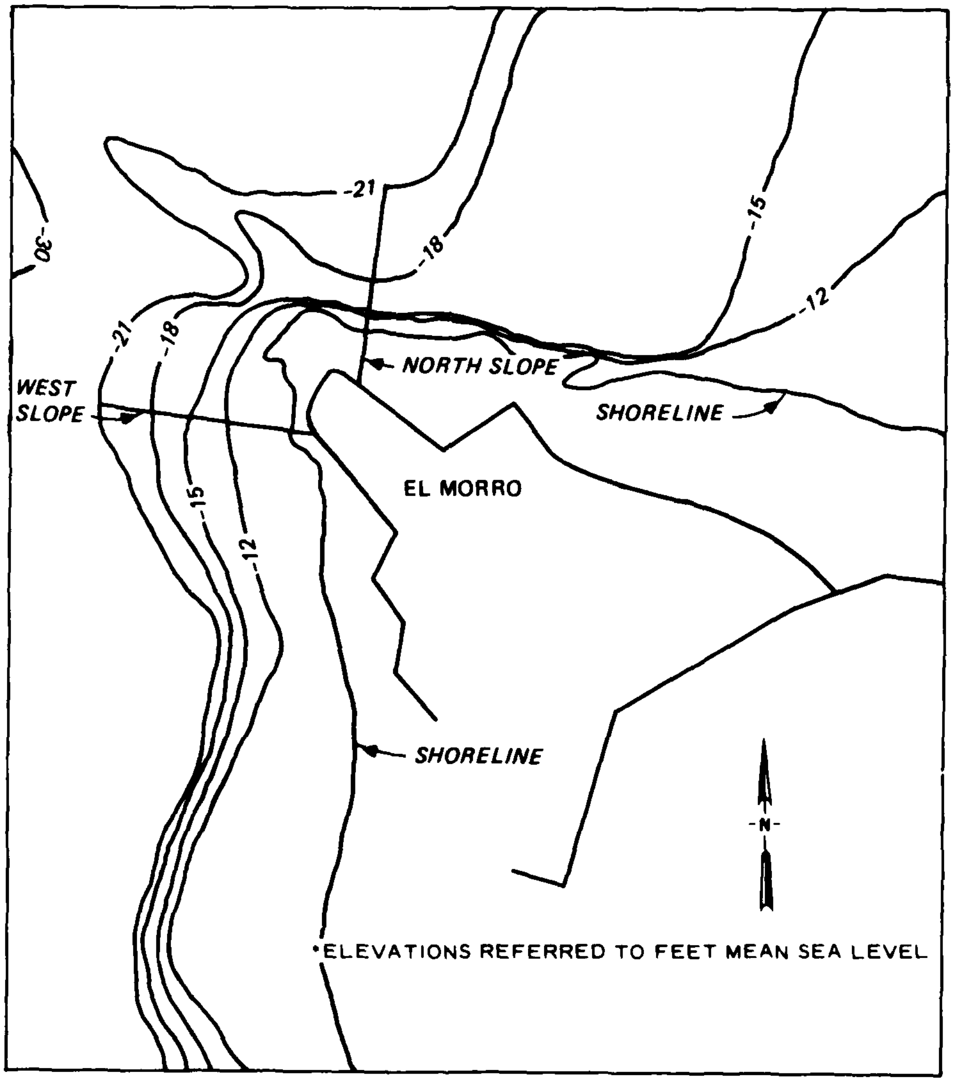
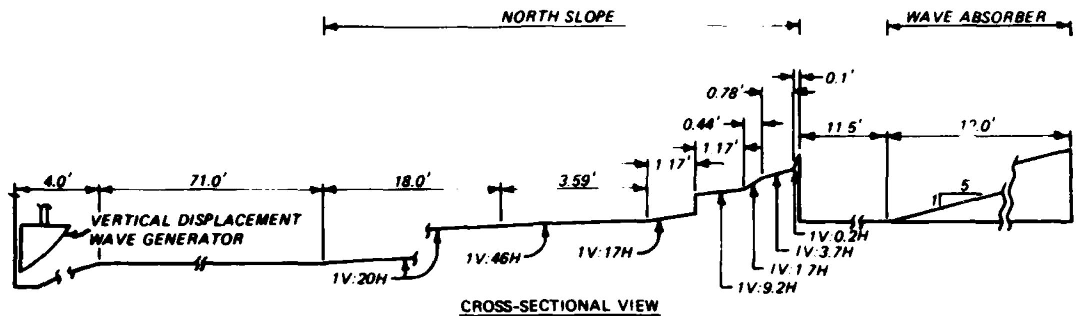
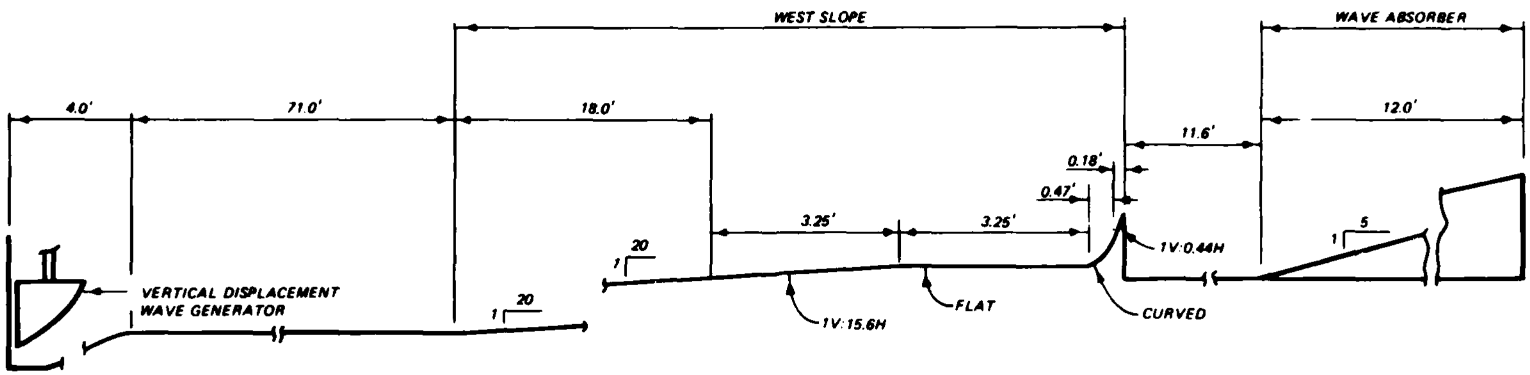


Figure 6. North and west slopes modeled for the 2-D stability tests of the offshore breakwater and north revetment and the west revetment, respectively



NOTE: ALL DIMENSIONS IN TERMS OF ACTUAL MODEL SIZE.

Figure 7. Flume geometry for the 2-D north slope tests



NOTE ALL DIMENSIONS IN TERMS OF ACTUAL MODEL SIZE.

Figure 8. Flume geometry for the 2-D west slope tests

3-D Test Flume

Flume geometry and wave generator

12. All the 3-D stability tests were conducted in a flume 35.5 ft wide, 3.5 ft deep (maximum depth), and 110 ft long (maximum length). The flume was equipped with a horizontal-displacement wave generator capable of producing monochromatic waves of various periods and heights. Figure 9 gives both a plan and a cross-sectional view of the 3-D test flume.

Test flume calibration

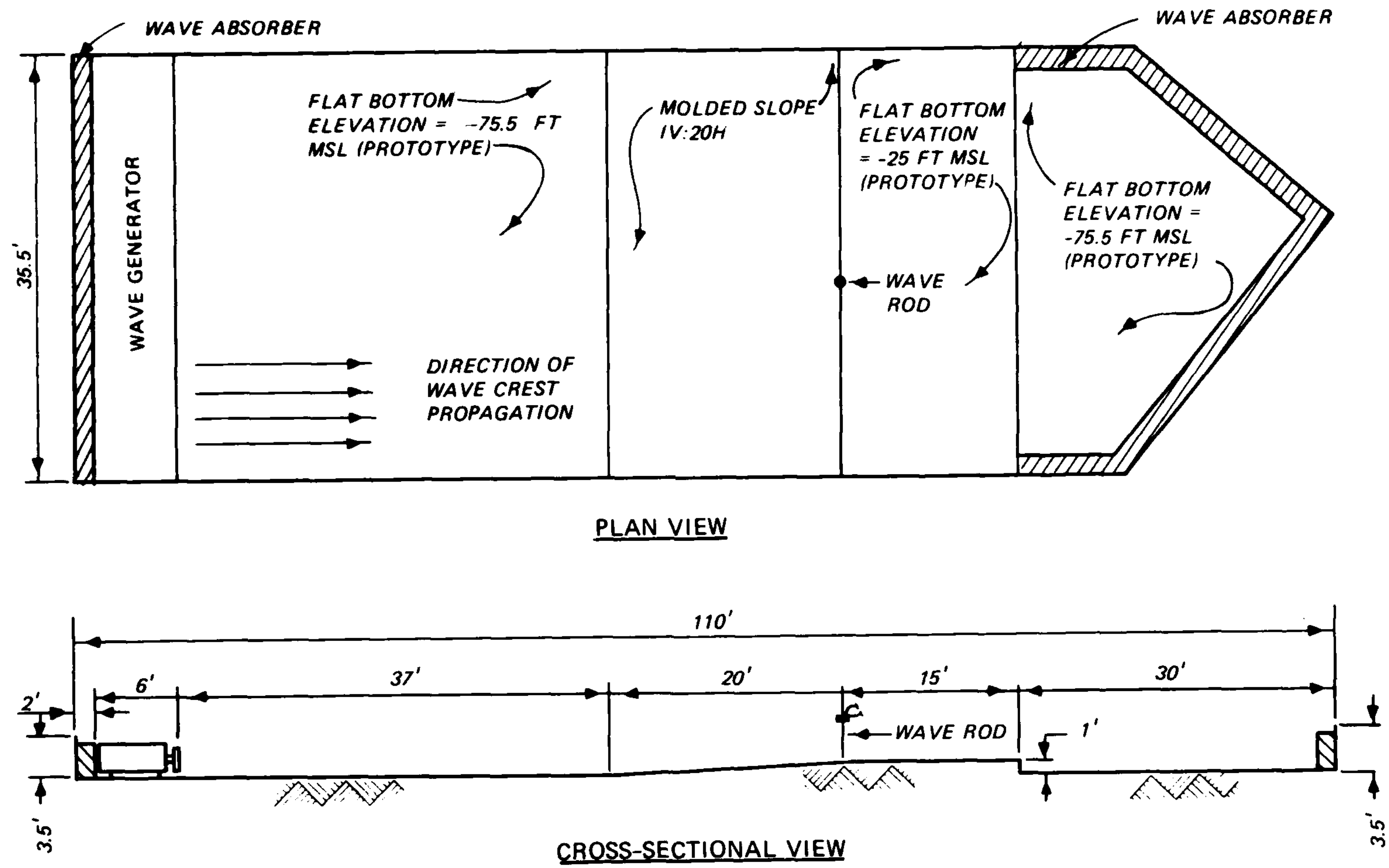
13. Calibration of the 3-D test flume was carried out in the same manner as previously described in paragraph 9, i.e., the wave heights were measured at the toe of the proposed breakwater head (el -25) without the breakwater in place. The top of the 1V-on-20H slope (Figure 9) represented the -25.0 ft contour in the prototype.

Modeling local bathymetry

14. The average local bathymetry in the area of the breakwater head and adjacent trunk was represented by a 1V-on-20H slope seaward of the -25.0 ft contour and a flat bottom landward of this contour (Figure 9). This allowed the 3-D test section to be reoriented for testing wave attack from various directions without requiring that the local bathymetry be remolded for each wave direction.

Selection of Test Conditions

15. The breakwater and revetments were tested for swl's of 0.0 and +1.9. These swl's were selected by SAJ to represent a normal tide condition and an extreme storm tide condition, respectively. The normal tide range at San Juan is +0.6 (mean high water) to -0.5 (mean low water). The initial test sections, both 2-D and 3-D, were tested to determine the worst breaking wave conditions that could occur on the structures for prototype wave periods of 7, 9, 11, 13, 15, and 17 sec at both swl's. Runup measurements for the north and west slopes were



NOTE: ALL DIMENSIONS IN MODEL TERMS UNLESS OTHERWISE NOTED.

Figure 9. Flume geometry and wave rod location for 3-D tests

taken for a range of incident wave heights at each of the prototype wave periods listed above.

2-D model - north slope

16. Model observations on the initial test section indicated that the 15- and 17-sec wave periods produced the worst breaking wave conditions at both the 0.0 and +1.9 swl's. For this reason, both the 15- and 17-sec wave periods were used for all subsequent full-length stability tests on the north slope. The 15-sec wave period was selected as the predominant wave period in the swl-wave versus time hydrographs used in the north slope breakwater and revetment stability tests due to its higher observed frequency of occurrence (relative to the 17-sec wave period). Hydrograph A (Plate 1 and Table 1) was used for the 2-D stability tests conducted on the north slope.

2-D model - west slope

17. During the initial tests of the unprotected, west slope revetment, model observations indicated that the 9-, 15-, and 17-sec wave periods produced the worst breaking wave conditions at both the 0.0 and +1.9 swl's. Hydrograph B (Plate 2 and Table 2) was used for the 2-D stability test conducted on the west slope.

3-D model

18. Incident wave directions of north, N30°W, and N72°W were selected for the 3-D stability tests of the offshore breakwater head and adjacent trunk. Initial testing of the offshore breakwater, for both the north and N30°W wave directions, revealed that for swl's of 0.0 and +1.9, the 15- and 17-sec wave periods produced the most critical depth-limited breaking wave conditions (Hydrograph 3D-A, Plate 3 and Table 3) on the breakwater structure. SAJ deemed that such wave conditions could exist from these directions; thus, Hydrograph 3D-A was used for all subsequent 3-D breakwater stability tests for incident wave directions of north and N30°W. Further, it was determined by SAJ that the maximum wave conditions which could occur from the N72°W direction resulted from 9- and 13-sec wave periods and would not exceed about 20 ft. Based on these data, Hydrograph 3D-B (Plate 4 and Table 4) was used for the 3-D

stability tests of the offshore breakwater head and adjacent trunk for incident waves from N72°W.

Methods of Reporting Damage

19. Detailed recordings of model observations were made during the conduct of all stability tests. The following list of adjectives, in order of increasing severity, was used to describe the activity taking place during the conduct of each stability test and resulting condition of the test sections at the conclusion of each test: (a) slight, (b) minor, (c) moderate, (d) significant, (e) major, (f) extensive. Use of these adjectives allowed some quantification of the severity and/or amount of rocking in place, onslope displacement, offslope displacement, and resulting damage accrued by the breakwater's and revetment's primary cover-layer protection. By using these descriptive adjectives and the photographs taken before and after testing, comparisons can be made between alternative breakwater and revetment designs.

PART III: TESTS AND RESULTS

Development of Plans

20. Based on guidance from SAJ, the prototype sea floor consists of hard rock with small outcroppings of sand. All of the model test plans assumed the breakwater sections were located on a nonscour rock bottom and did not require breakwater bedding and apron material. Where prototype breakwater sections are sited on sand outcroppings, it is recommended that bedding and toe protection material be used to prevent undermining and excessive settling of the structure.

2-D model - north slope

21. Seven plans for the offshore breakwater and protected revetment were tested (undistorted scale, 1:38.5) on the north slope bathymetry. Five plans (Plans N-1, N-2, N-3, N-3-A, and N-4, Plates 5-9, respectively) used two layers of armor stone as the primary armor protection on the offshore breakwater. Plans N-5 and N-6 (Plates 10 and 11, respectively) used two-layer, dolos armor protection. Plan N-7 (Plate 12) was used for a check test of the unprotected north slope revetment. All of the north slope revetment plans, both protected and unprotected, used two-layer armor-stone protection.

22. Two special tests were conducted to look at the severity of wave attack that could occur on the construction trestle prior to and during the construction of the offshore breakwater and revetment on the north slope. The trestle was tested on the north slope without the breakwater and revetment in place (Plate 13) and then exposed to the same wave attack with a portion of the breakwater and revetment in place (Plate 14).

2-D model - west slope

23. Four unprotected revetment plans were tested (undistorted scale 1:38.5) on the west slope bathymetry. All four designs (Plans W-1, W-2, W-3, and W-4, Plates 15-18, respectively) used two-layer, armor-stone protection.

3-D model

24. Plan 3D-1 (Plate 19) was used for all 3-D stability tests (undistorted scale 1:50.5). The offshore breakwater head and adjacent trunk were constructed with the same weight armor stone as that tested on Plans N-3 and N-3-A (north slope, 2-D tests).

Description of Test Plans and Test Results

2-D model - offshore breakwater and protected north slope revetment

25. Plan N-1 (Plate 5 and Photos 10 and 11) consisted of an offshore breakwater and protected north revetment. The breakwater was constructed to a crown elevation of +2.0 using side slopes of 1V on 2H and the seaside toe of the breakwater located at the -21.0 ft contour. Two layers of 93,600-lb armor stone were randomly placed in a two-layer system over the 4,680-lb core material. The north revetment was constructed to an elevation of +5.1 using a 1V on 3H slope. Two layers of randomly placed, 2,100-lb armor stone covered the 210-lb core material. After exposure to Hydrograph A (Plate 1 and Table 1), the breakwater showed no damage with only minor rocking of a few of the armor stone occurring throughout the test hydrograph. The crown of the north revetment was lowered approximately 3 to 4 ft and widened to approximately 30 ft causing exposure of the vertical ledge to transmitted wave action. Photos 12 and 13 show Plan N-1 at the conclusion of Hydrograph A. Plan N-1 was rebuilt and again exposed to Hydrograph A and similar damage was accrued on the north revetment. During the repeat test, one armor stone was displaced from the sea-side slope to the toe of the breakwater during Step 2 of Hydrograph A. No other armor-stone displacement occurred on the breakwater for the remainder of the hydrograph but minor rocking of a few armor stone did occur throughout the test. Photos 14 and 15 show Plan N-1 after the repeat testing of Hydrograph A.

26. Plan N-2. With the breakwater armor stone on Plan N-1 having proved to be more than adequate and the revetment armor stone inadequate

for the design conditions of Hydrograph A, testing of Plan N-2 (Plate 6 and Photos 16 and 17) was initiated in an effort to find a more economical design for the offshore breakwater and a stable design for the protected north revetment. The crown elevations, crown widths, and side slopes used on Plan N-2 were identical to Plan N-1, but the average weights of the breakwater armor stone and core material were reduced relative to Plan N-1, while the average weights of the revetment materials were increased. On the offshore breakwater, two layers of 67,800-lb armor stone were randomly placed over the 3,390-lb revetment core material. Two layers of 13,500-lb armor stone were randomly placed over the 1,360-lb revetment core material. After exposure to Hydrograph A, the structure had accrued only minor damage to both the breakwater sea-side toe, with three armor stones displaced, and the revetment, with eight armor stones displaced. Damage had stabilized at the conclusion of Hydrograph A and had not adversely affected either the structural or functional integrity of the structure. Photos 18 and 19 show Plan N-2 at the conclusion of Hydrograph A. After rebuilding Plan N-2, Hydrograph A was again tested and results of this test were very similar to those of the first testing. Photos 20 and 21 show results of the repeat test of Hydrograph A on Plan N-2. Both the offshore breakwater and north revetment sustained minor damage during this test. The amount of damage was slightly less than what occurred during the first test with only one armor stone displaced on the sea-side toe of the breakwater and six armor stones displaced on north revetment. Damage had stabilized at the conclusion of the test.

27. During testing of Plans N-1 and N-2 with Hydrograph A, it was observed that the area between the offshore breakwater and the protected north revetment tended to pond water during wave attack. It was felt that this ponded water could be giving added protection to the revetment armor stone; and this ponding may or may not occur in the prototype depending on the area of the breakwater, angle of wave attack, and other storm parameters. In addition, it appeared that the backrush of water out of this ponded area and over the crown of the breakwater was affecting the breaking action of the test waves. It was

not obvious whether this effect was important (either increasing or decreasing) to stability of the breakwater. In an effort to alleviate the ponding of water in this area, a 6- by 9-in. culvert, as indicated in Photos 22 and 23, was cut in the wall between the San Juan test flume and the test flume adjacent to it. These two flumes share a common wave generator and also maintain the same swl, but since no structure or overbank was installed in the adjacent flume, some lateral release (flow) of the ponded water was made possible.

28. Plan N-2 was rebuilt and exposed to Hydrograph A with the culvert open. The culvert reduced, but did not totally alleviate, the water ponding between the breakwater and revetment. Stability of the breakwater and revetment was very similar to the results without the culvert. The first testing of Plan N-2 with the culvert open resulted in minor damage to both the breakwater and revetment with five armor stones displaced in each area (Photos 22 and 23). After the test section was rebuilt, a repeat testing of Hydrograph A was conducted on Plan N-2 with the culvert open. The breakwater and revetment sustained minor damage during this repeat test. Two to three armor stones were displaced on the sea-side toe of the breakwater and 12 to 14 armor stones were displaced on the revetment (Photos 24 and 25). The damage had stabilized at the conclusion of Hydrograph A on both tests. Both the functional and structural integrity of the breakwater and revetment was maintained during both testings of Hydrograph A with the culvert open. Although the effects on stability with and without the culvert open were not significant, it was deemed that tests with the culvert open best represented the prototype; thus, all subsequent tests were conducted with the culvert open.

29. During the testing of Plan N-2 wave action Photos 26-33 were taken of Steps 1-4 of Hydrograph A. Both side and sea-side views were taken to show the wave form and impact areas on the breakwater.

30. Plan N-2 runup. Stability testing for Plan N-2 was concluded and runup measurements were made landward of the north revetment armor stone for 7- to 17-sec wave periods and 5- to 24-ft wave heights for swl's of 0.0 and +1.9 with the culvert open. Since the roughness

of the prototype slope was not known, the north slope between the +5.1 and +28 ft contours was initially constructed with a smooth finish (see overbank in Photo 33). It was felt that if the runup was acceptable on this smooth slope then it would be acceptable in the prototype where a greater roughness exists. Preliminary tests showed runup on the smooth slope to be quite high; thus, it was decided that a closer representation of an assumed roughness of the north slope should be considered. The upper slope was roughened (see overbank in Photo 34) and runup tests were repeated for Plan N-2. These runup data are given in Table 5 and Plates 20 and 21. While it is difficult to compare these runup data with those from the 3-D wave action model (Bottin 1979) (i.e., test waves for 2-D stability tests were measured at the -21 ft contour while those for the 3-D wave action model were measured at the 60 ft contour), it appears that the 2-D runup values are somewhat larger. Some of the reasons that this would be expected are:

- a. Reproduction of slope roughness and accuracy of measurements between the 1:75 and 1:38.5 scales may be different.
- b. The 3-D model (actual contours) allowed refraction and diffraction (i.e., transmittal of energy laterally or to the sides) while the 2-D model (idealized slope) confined all wave energy between the flume walls.
- c. The buildup of water behind the breakwater probably was not as great in the 3-D model since water could escape to both sides (i.e., flow out both ends of the breakwater).
- d. The still-water levels tested were different (i.e., +1.1 for the 3-D wave action model and 0.0 and +1.9 for the 2-D stability model).

31. Plan N-3. With Plan N-2 showing very minor damage after exposure to Hydrograph A, tests were initiated for Plan N-3 (Plate 7 and Photos 34 and 35) to find a more economical design for the offshore breakwater and revetment for the north slope. The geometrical size and shape of Plan N-3 were identical to Plans N-1 and N-2. Plan N-3 consisted of smaller (relative to Plan N-2) armor stone and core material on both the offshore breakwater and revetment. Two layers of 55,500-lb armor stone were placed in a random manner over the 2,775-lb breakwater

core material. The revetment was constructed with two layers of randomly placed 9,360-lb armor stone over the 935-lb core material. After exposure to Hydrograph A, Plan N-3 showed minor to moderate damage (Photos 36 and 37). Onslope displacement, rocking in place, and downslope slippage of armor stone on the sea-side face and crown of the breakwater occurred throughout testing with Hydrograph A. The armor-stone movement for Plan N-3 increased relative to Plan N-2; however, the average elevation of the breakwater crown was not lowered by the movement that occurred. The beach-side slope and toe of the breakwater were stable with only one armor stone being unseated during the test. Revetment armor stone exhibited more downslope displacement and rocking in place than had occurred for Plan N-2. A slight lowering (1 to 2 ft) occurred in a few areas of the revetment crown but this was not a uniform lowering along the entire crown. Plan N-3 was rebuilt and a repeat test using Hydrograph A was conducted. The resulting damage, Photos 38 and 39, to the offshore breakwater and revetment was very similar to the first test. A slightly higher amount of onslope displacement and rocking in place of the breakwater's sea-side armor stone was observed during this test. During both testings of Plan N-3, damage to the structure had stopped before the end of the hydrograph.

32. The amount of onslope displacement that occurred on the sea-side slope, crown of the breakwater, and revetment slope of Plan N-3 seemed to indicate that any further reduction in armor-stone weight in these areas would most likely result in significant damage to the structure when exposed to Hydrograph A. The lower portion of the breakwater's beach-side slope showed no instability during testing of Plan N-3 and it was felt that the armor-stone weight in this area could be reduced without affecting the overall stability of the structure.

33. Plan N-3-A. Tests were initiated on Plan N-3-A (Plate 8 and Photos 40 and 41) to optimize the design of Plan N-3. Plan N-3-A was identical to Plan N-3 except for the breakwater's beach-side slope below the -6.0 elevation. The tow layers of 55,500-lb armor stone used in this area on Plan N-3 were replaced with 9,360-lb armor stone. Like Plan N-3, Plan N-3-A accrued minor to moderate damage to the breakwater

crown and sea-side slope when exposed to Hydrograph A (Photos 42 and 43). A moderate amount of onslope displacement occurred throughout the hydrograph. Two armor stones were displaced from the crown of the breakwater onto the revetment area. Both the 55,500- and 9,360-lb beach-side armor stone exhibited only minor movement. The revetment slope showed the same slight downslope displacement and spot lowering of the +5.1 crown elevation as had occurred for Plan N-3. Plan N-3-A was rebuilt and again exposed to Hydrograph A (Photos 44 and 45). The beach-side slope of the breakwater and the revetment slope showed the same type and amount of displacement as had occurred during the first testing. The sea-side slope and crown of the breakwater accrued more damage during this testing of Plan N-3-A than had occurred during the first testing of Plan N-3-A and either testing of Plan N-3. The functional and structural integrity of the breakwater was not lost, but the resulting damage was very close to exceeding the no-damage design criteria.

34. Plan N-3-A, runup. At the conclusion of the stability tests for Plan N-3-A, runup measurements were taken for the +1.9 swl. The runup measured with Plan N-3-A showed no net increase or decrease relative to heights measured for Plan N-2. These data are tabulated and plotted in Table 6 and Plate 22, respectively.

35. Plan N-4. Testing was initiated for Plan N-4 (Plate 9 and Photos 46 and 47) to determine if any further reduction in the armorstone weight on the breakwater and revetment would result in total instability of the structure. The size and geometry of Plan N-4 were identical to all previous plans with the only changes being in the weights of the armor stone and core materials used. The offshore breakwater was constructed using two layers of randomly placed 45,825-lb armor stone from the sea-side toe to the -6.0 elevation on the beach-side slope of the breakwater. Two layers of 6,100-lb armor stone were placed, in a random manner, from the +5.1 crown of the revetment to the -6.0 elevation on the beach-side slope of the breakwater. Core material weights of 2,290-lb and 610-lb were used for the breakwater and revetment, respectively. Continuous onslope displacement, rocking in place,

and unseating of the breakwater's sea-side and crown armor stone were observed throughout the first testing with Hydrograph A (Photos 48 and 49). Major downslope movement and flattening of the revetment crown occurred. The majority of the revetment damage occurred during testing of the 17-sec wave period at both swl's. Three armor stones were displaced over the crown of the breakwater and onto the revetted area. One sea-side armor stone was displaced down the sea-side slope and off the structure. Major changes occurred on the breakwater's sea-side slope and close comparison of Photos 47 and 49 (before and after testing sea-side photographs for the first test section of Plan 4, respectively) shows that a majority of the sea-side armor stone have moved and changed their orientation and a general loosening of the slope is obvious. Several of the armor stones were turned completely over during the test, yet most of these did not move completely out of their original position. Though not obvious in the photographs, some downslope slumping of the seaward face of the breakwater occurred. This resulted in a slight lowering of the crown seaward of the center line of the structure and thus a reduction of the original 40-ft width of +2.0 crown elevation. The high degree of armor-stone movement observed during the first testing with Hydrograph A for Plan 4 showed that the breakwater and revetment have a high potential for accruing major damage. A repeat testing of Plan 4 with Hydrograph A resulted in major damage to both the breakwater and revetment (Photos 50 and 51). The revetment crown was lowered 4 to 5 ft exposing the upper portion of the vertical ledge to wave attack. One armor stone was displaced from the breakwater crown onto the revetted area. Eight armor stones were displaced off the breakwater's sea-side slope. Although not displaced off the structure, a majority of the cover-layer armor stone on the sea-side of the breakwater showed a high degree of movement and rocking in place throughout the test. Seaward movement of the breakwater toe caused the breakwater slope to slump in two main areas as indicated by the dashed lines in Photo 51. Damage to the structure had not stabilized at the conclusion of Hydrograph A. The breakwater had accrued more damage than was acceptable and for this reason the test was not extended.

36. Plots of armor-stone weights versus damage were prepared to aid in comparing the stabilities of Plans N-1, N-2, N-3, N-3-A, and N-4 when exposed to the wave and swl conditions of Hydrograph A. Plate 23 is a plot of breakwater armor-stone weight versus estimated number of stone rocking in place and/or displaced onslope, average number of stone displaced offslope, and overall observed damage on the offshore breakwater. Plate 24 is a plot of revetment armor-stone weight versus maximum deteriorated crown width of revetment, estimated maximum distance revetment crown was lowered, and overall observed damage on the protected north revetment.

37. 2-D model, trestle tests. Upon completion of the armor-stone breakwater and revetment stability tests, the construction trestle was installed on the north slope without the revetment and offshore breakwater (Plate 13). The trestle was tested with 7- to 17-sec wave periods using a range of wave heights at both the 0.0 and +1.9 swl's (Table 7) to determine if these conditions would be potentially damaging to the trestle. The tests indicated that incident wave heights 10 ft and lower (measured during calibration at -21.0 ft contour) should not endanger the trestle. For incident wave heights greater than 10 ft, the incident and reflected wave heights combine to create conditions that could be damaging to the trestle. For wave periods from 8.0 to 9.0 sec, these conditions occur very close to the trestle and create waves that strike directly underneath the trestle decking. For wave periods greater than 9.0 sec, the incident and reflected waves combine (to produce the maximum water elevation) seaward of the trestle and create wave conditions that impact on the front face or break onto and/or over the top of the trestle decking. Plots in Plates 25 and 26 show where the incident and reflected waves combine (to produce the maximum water elevation) relative to the vertical ledge for 7.0- to 17.0-sec wave periods at swl's of +1.9 and 0.0, respectively. The plots also show the maximum water-surface elevations at these locations for incident wave heights of 5.0 to 20.0 ft. Photos 52 and 56 show two of the wave and swl conditions that pose no danger to the construction trestle.

Photos 53, 54, 55, and 57 show four of the wave and swl conditions that could be damaging to the trestle.

38. During construction of the prototype structures, the breakwater and trestle will be adjacent to one another. The question arose regarding whether or not this would create wave conditions that could be potentially more damaging to the trestle than those that occur without the breakwater and revetment adjacent to the trestle. Plan N-3-A and the trestle were constructed on the north slope, as shown in Plate 14 and Photos 58 and 59. Plan N-3-A and the trestle were exposed to the same wave and swl conditions described in paragraph 37 (Table 7). The combined incident and reflected waves for the various wave periods occurred at approximately the same locations as observed with only the trestle in place; however, due to the difference in reflection characteristics between the vertical ledge and breakwater and dissipation of wave energy by the breakwater, the locations were not as well defined and the maximum water-surface elevations were not as high. The conditions observed were not as severe as with the trestle alone, but incident wave heights above 10.0 ft still caused conditions that could possibly be damaging to the trestle. Photos 60-73 show side and sea-side views for a range of wave conditions at 0.0 and +1.9 swl's. Comparison of these photos with Photos 52-57 for the same wave and swl conditions shows that the wave attack on the trestle was less severe when the breakwater and revetment were adjacent to the trestle.

39. Plan N-5. With the possibility of the quarry not being able to yield the size of armor stone needed for stability of the offshore breakwater, tests were initiated with Plan N-5 (Plate 10 and Photos 74 and 75) in an effort to find a stable dolos-armored offshore breakwater design for the wave and swl conditions of Hydrograph A. The overall geometric size of Plan N-5 was identical to all previously described breakwater and revetment plans for the north slope. The revetment core and armor-stone sizes were identical to Plan N-3-A and the revetment armor stone extended up the beach-side slope of the breakwater to the -6.0 elevation. Two layers of randomly placed 15,135-lb dolosse were placed over the 3,027-lb breakwater core material from the sea-side toe

to the -6.0 elevation on the beach side of the breakwater. After exposure to Hydrograph A, the offshore breakwater showed significant damage with a total of 26 dolosse displaced from the crown and beach-side slope onto the revetment (Photos 76 and 77). The displacement of dolosse from the breakwater crown caused some lowering of the original +2.0 crown elevation. This crown lowering allowed larger amounts of wave energy to reach the revetment causing some minor spot damage along the revetment crown. The breakwater's sea-side slope showed moderate onslope displacement but no significant damage resulted. At the conclusion of Hydrograph A, damage to the dolos armoring on the crown and beach-side slope of the breakwater had not stabilized. The revetment armor stone showed minor damage which had stabilized by the end of the test. The damage to the dolos armor exceeded the allowable amount for an acceptable design and the test was not extended.

40. Plan N-6. In an effort to find a stable dolos armor design for the offshore breakwater, tests were conducted on Plan N-6 (Plate 11 and Photos 78 and 79). Plan N-6 was identical to Plan N-5 except for the dolos armor and core weights which were increased to 21,830 and 4,366 lb, respectively. After exposure to Hydrograph A, the offshore breakwater's dolos armor showed moderate damage on the crown and beach-side slope with a total of seven dolosse having been displaced onto the revetment armor stone. The breakwater's sea-side slope and the revetment armor stone had accrued only minor damage. All damage on the structure had subsided by the end of the test and the condition of the structure can be seen in Photos 80 and 81. Plan N-6 was rebuilt and once again exposed to the wave and swl conditions of Hydrograph A. Results of this repeat test were very similar to the first test with six dolosse displaced onto the revetment from the crown and beach-side dolos armor areas. After-test Photos 82 and 83 show that the revetment armor stone and the breakwater's dolos armor on the sea-side slope had sustained only minor damage.

41. During the testing of Plan N-6, wave action photographs were taken showing the wave attack of Steps 1-4 of Hydrograph A (Photos 84-91). These side and sea-side views were taken to show the impact

area and form of the breaking waves.

42. Plan N-6, runup. Runup measurements were made for Plan N-6 landward of the north revetment for 7- to 17-sec wave periods and 5- to 24-ft wave heights (measured during calibration at the -21.0 ft contour) for swl's of 0.0 and +1.9. The exact test conditions and the corresponding range of runup are given in Table 8. Plates 27 and 28 show maximum runup as a function of wave heights. These runup measurements are, on the average, slightly lower than the values measured on Plans N-2 and N-3-A for the same wave and swl conditions.

43. Near the conclusion of Plan N-6 testing, WES was notified by SAJ that at least one quarry within the San Juan area would yield up to 30-ton armor stone, and for that reason, SAJ was no longer considering the use of dolos armoring for the offshore breakwater. SAJ stated that WES should finish testing Plan N-6 but that no further efforts should be made by WES to optimize the dolos-armored, offshore breakwater design.

2-D model - unprotected north slope revetment, Plan N-7

44. A portion of the revetment on the north slope will be unprotected, that is, no offshore breakwater will be constructed seaward of the revetted area. Before the north slope topography was removed from the test flume, one check test of Plan N-7, Plate 12, was conducted to see if 24,530-lb revetment armor stone would be stable for the worst depth-limited breaking wave condition that could occur at an swl of +1.9 and a wave period of 15.0 sec. After 3.0 hr of 15.0-sec, 16.5-ft breaking waves (measured at the -18.2 ft contour) at an swl of +1.9, the revetment armor stone showed minor to moderate damage (Photos 92 and 93). One armor stone was displaced off the structure. The revetment crown showed some spot lowering (1 to 2 ft) of the original +2.0 elevation due to the reorientation and downslope movement of a few armor stones. At the end of the test, damage to the revetment had subsided and it was concluded this plan would provide adequate protection.

2-D model - unprotected west slope revetment

45. Plan W-1 (Plate 15 and Photos 94 and 95) consisted of a

revetment plan for the unprotected west slope just south of the western tip of the offshore breakwater. The revetment was constructed using a 1V-on-3H slope between the +6.0 and -7.0 elevations. A horizontal berm extended 10 ft seaward from the base of the 1V-on-3H slope and then descended on a 1V-on-1.5H slope to the -13.0 toe elevation at the outer edge of the revetment. Core material averaging 1,033 lb in weight was overlaid with two layers of randomly placed primary armor stone. The armor layers were composed of stone having an average individual weight of 10,330 lb. After exposure to Hydrograph B (Plate 2 and Table 2), Plan W-1 showed no damage; and the only visible movement was minor rocking of three to four armor stones throughout the test. Photos 96 and 97 show the condition of the structure at the end of the first test. The test section was rebuilt and once again exposed to the wave and swl conditions of Hydrograph B. After-test Photos 98 and 99 show that the results of the second testing of Plan W-1 were very similar to those of the initial test.

46. Plan W-2. In an effort to optimize the design of the unprotected west revetment, tests were initiated on Plan W-2 (Plate 16 and Photos 100 and 101). The core and armor-stone weights were reduced to 700 and 7,000 lb, respectively. A two-layer, randomly placed armor-stone protection was used; and the overall size and shape of Plan W-2 were identical to Plan W-1. Exposure of Plan W-2 to the design conditions of Hydrograph B resulted in only minor spot damage along the revetment crown. As shown in after-test Photos 102 and 103, the remainder of the structure showed no obvious damage. Plan W-2 was rebuilt and exposed to the wave and swl conditions of Hydrograph B once again. During the repeat testing, Plan W-2 accrued a similar amount of spot damage to the revetment crown as had occurred during the initial test. Photos 104 and 105 show the condition of the revetment at the conclusion of the repeat test.

47. Plan W-2, runup. Runup measurements for Plan W-2 on the west slope were made for wave periods from 7 to 17 sec and a range of wave heights at an swl of +1.9. These data are tabulated in Table 9 and presented graphically in Plate 29.

48. Plan W-3. With only minor damage having occurred on Plan W-2, tests were initiated on Plan W-3, (Plate 17 and Photos 106 and 107). The primary armor stone (4,000 lb) was randomly placed in a two-layer system over 400-lb core material. As with Plan W-2, the overall size and shape of Plan W-3 were identical to Plan W-1. Exposure to the design conditions of Hydrograph B resulted in moderate damage along the revetment crown. No offslope displacement of armor stone occurred, but a minor to moderate amount of onslope movement was observed throughout the test. Photos 108 and 109 show the condition of Plan W-3 at the end of the first test. After-test Photos 110 and 111 of the repeat testing of Plan W-3 using Hydrograph B show results very similar to the first test. More onslope movement was observed during the repeat test, but this movement did not result in any offslope displacement. The movement witnessed during Plan W-3 is more than that usually observed for the no-damage stability criteria; however, since the end results did not indicate offslope deterioration, this plan was considered marginally stable.

49. Plan W-4. Tests were initiated on Plan W-4 (Plate 18 and Photos 112 and 113) to determine if any further reduction in the armor-stone weight on the unprotected west revetment would result in total instability of the structure. The overall size and geometry of Plan W-4 were identical to all previous revetment plans tested on the west slope. Both the armor-stone and core material weights were reduced on Plan W-4. Two layers of 2,000-lb armor stone were placed in a random manner over the 200-lb core material. After exposure to Hydrograph B, Plan W-4 showed moderate to significant damage to the revetment crown. Some areas of the crown had been lowered as much as 5 ft. A significant amount of onslope movement of the 2,000-lb armor stone occurred throughout the test, but this did not result in any appreciable offslope displacement. Damage to the revetment crown had subsided at the conclusion of the test, and the after-test condition of Plan W-4 is shown in Photos 114 and 115. After being rebuilt, Plan W-4 was once again exposed to the wave and swl conditions of Hydrograph B. The revetment armor stone showed the same amount of onslope movement, but the

resulting damage to the revetment crown was less severe than what had occurred during the first test. Damage had stabilized at the conclusion of the test. Photos 116 and 117 show the condition of the structure after testing.

50. Plate 30 is a plot of armor-stone weight versus estimated maximum distance crown was lowered, onslope armor-stone movement, and overall observed damage on the unprotected west revetment. This combined plot and bar graph was prepared as an aid for comparing the stabilities of Plans W-1, W-2, W-3, and W-4 (the unprotected west revetment) when exposed to the wave and swl conditions of Hydrograph B.

3-D model - west head of offshore breakwater

51. Wave direction N30°W. Plan 3D-1 (Plate 19 and Photos 118-120) consisted of the west end of the offshore breakwater including the head and approximately 314 ft of the trunk. The breakwater was constructed using side slopes of 1V on 2H from the -25.0 toe elevation to the +2.0 crown elevation. The breakwater trunk had a crown width of 40 ft, and the radius of curvature of the crown on the breakwater head was 20 ft. Two layers of 55,350-lb armor stone were placed over the 2,768-lb core material. Random armor-stone placement was used as had been used on all of the 2-D stability test sections. The armor-stone and core material weights are the same as those found to be stable on the 2-D test Plans N-3 and N-3-A. The 3-D test section was exposed to the wave and swl conditions of Hydrograph 3D-A (Plate 3 and Table 3) for incident waves from N30°W (Plate 31). At the conclusion of the first test, all damage to the breakwater had stopped and the structure showed only minor damage. A total of six armor stones had been displaced off the structure's trunk, and no offslope displacement had occurred on the head (Photos 121-123). The majority of the trunk and all of the head showed only minor rocking and onslope movement. This amounted to approximately five to six armor stones rocking in place or moving a small distance onslope. The sea-side trunk adjacent to the breakwater head, indicated in Photo 121 as "Area A," showed moderate rocking in place and onslope movement of armor stone. Throughout the

first test this was the most active portion of the breakwater, but only very moderate, stabilized damage occurred in portions of "Area A." Plan 3D-1 was rebuilt and was once again exposed to wave attack from N30°W using the design conditions of Hydrograph 3D-A. Results of this test were very similar to those of the first test. A total of six armor stones, four from the trunk and two from the head, were displaced off the structure. Minor onslope rocking in place and movement occurred over the entire breakwater throughout the majority of the test, but all damage had stabilized by the conclusion of the hydrograph. Once again, "Area A" (Photo 124) showed more activity than any other portion of the breakwater; but as a whole, the breakwater showed only minor damage after exposure to Hydrograph 3D-A (Photos 124-126).

52. Wave direction north. The structure was reoriented for wave attack from the north (Plate 32), rebuilt (Photos 127-129), and exposed to Hydrograph 3D-A. A total of 10 armor stones were displaced off the structure and an additional 5 armor stones were displaced onslope. All of the armor-stone displacement, both onslope and offslope, occurred on the trunk of the breakwater, except for one armor stone which was displaced on the breakwater head. After-test Photos 130-132 show that the breakwater had accrued a moderate amount of damage. "Area B," as indicated in Photo 130, appeared to be the most active portion of the breakwater. The damage had stabilized at the conclusion of the test and the functional and structural integrity of the breakwater was still intact. The structure was rebuilt, and a repeat test using Hydrograph 3D-A was conducted for incident waves from the north. As seen in after-test Photos 133-135, the structure sustained only slight damage during this test. Very minimal onslope movement and rocking of armor stone occurred during this test. Two armor stones were displaced off the structure, one from the head and one from the trunk. All damage to structure had subsided at the end of the test.

53. Wave direction N72°W. Plan 3D-1 was reoriented for wave attack from N72°W (Plate 33). After being rebuilt (Photos 136-138) the structure was exposed to the wave and swl conditions of Hydrograph 3D-B (Plate 4 and Table 4). As seen in the after-test Photos 139-141, a

total of four armor stones from the crown and beach-side slope of the breakwater head were displaced off the structure. This resulted in minor to moderate spot damage to the crown of the breakwater (indicated in Photos 140 and 141). Two additional armor stones were displaced on the beach-side slope of the trunk but this did not result in any damage to the breakwater. All damage had stopped early within the last step of the hydrograph. The structure was rebuilt and once again exposed to Hydrograph 3D-B. Photos 142-144 show the condition of the structure at the end of the test. A total of five armor stones were displaced off the structure, two from the head and three from the beach-side slope of the trunk. Two additional armor stones were displaced onslope on the trunk. At the end of the test all movement had stopped and the breakwater showed only slight damage, as indicated in Photo 144. Photo 145 shows the wave action on the breakwater head and trunk for a breaking wave with an incident wave direction of N72°W at an swl of 0.0.

PART IV: CONCLUSIONS

54. Based on the tests and results reported herein, it is concluded that:

a. Offshore breakwater and protected north slope revetment. For the depth-limited breaking wave conditions produced on the north slope for swl's of 1.0 and +1.9 (Hydrograph A):

   (1) Plan N-1 is a more than adequate design for the offshore breakwater trunk.

   (2) Plan N-1 is not an adequate design for the protected north revetment.

   (3) The offshore breakwater trunk and protected north revetment of Plan N-2 are adequate designs.

   (4) The offshore breakwater trunk and protected revetment of Plans N-3 and N-3-A are satisfactory designs. Some minor displacement could occur on both the revetment and the sea-side slope of the breakwater trunk with either plan but the functional integrity of the armor protection should not be affected. Plan N-3-A is considered the optimum design relative to stability.

   (5) Plan N-4 is not an adequate design for either the protected north revetment or the breakwater trunk.

   (6) The dolos armoring on Plan N-5 is not an adequate design.

   (7) The dolos armoring on Plan N-6 is a marginally acceptable design.

   (8) The armor stone on the beach-side slope of the breakwater trunk and on the protected north revetment of Plans N-5 and N-6 is an adequate design.

b. Trestle tests. For wave periods ranging from 7 to 17 sec at swl's of 0.0 and +1.9, incident wave heights (measured on the north slope at the -21.0 ft contour) greater than 10.0 ft could create conditions that would be potentially damaging to the construction trestle. These conditions appear to be more severe with only the trestle in place, but they also occurred when the trestle, breakwater, and protected north revetment were on the north slope concurrently.

c. Runup tests. Although influenced by 2-D assumptions and effects (see paragraph 30, a through d) wave runup values obtained for Plans N-2, N-3-A, N-6, and W-1 are resonable

and valid considerations for prototype design. When exposed to the same wave and swl conditions on the north slope, runup values on the protected stone revetments behind breakwater designs of stone armor (Plans N-2 and N-3-A) were about the same, but were slightly less for the dolos armor design (Plan N-6).

d. Unprotected north slope revetment. The armor stone on the unprotected north revetment, Plan N-7, is an adequate design for the 15-sec, 16.5-ft breaking wave conditions produced on the north slope at a +1.9 swl.

e. Unprotected west slope revetment. For the breaking wave conditions produced on the west slope for swl's of 0.0 and +1.9 (Hydrograph B):

(1) Plans W-1 and W-2 are completely adequate designs for the unprotected west revetment.

(2) Plan W-3 is a marginally acceptable design for the unprotected west revetment.

(3) Plan W-4 is not an acceptable design for the unprotected west revetment.

f. Three-dimensional head and trunk tests. For the depth-limited breaking wave conditions produced at swl's of 0.0 and +1.9 (Hydrograph 3D-A), Plan 3D-1 is an adequate design for the head and adjacent trunk of the offshore breakwater for incident waves from the north and N30°W. Plan 3D-1 is also adequate for the maximum proposed wave conditions (Hydrograph 3D-B) for an incident wave direction of N72°W.

PART V: DISCUSSION AND RECOMMENDATIONS

55. If at some future date, for reasons not apparent at this time, dolosse are seriously considered for armoring the offshore breakwater, additional stability tests should be conducted. Additional 2-D stability tests of the breakwater trunk and some 3-D stability tests of the breakwater head and adjacent trunk would be needed to ensure a sound dolos engineering design for the offshore breakwater.

56. Although the breaking wave conditions and revetment developed in the model for the unprotected west slope are appropriate and reasonable for the area simulated in the model, they are dependent upon the particular topography (Plate 15), wave approach (Figure 6), and wave conditions specified by SAJ. If these parameters should change as the revetment design proceeds south along the west slope, consideration should be given to recalculating the armor size according to the change in wave height (i.e., steeper contours occurring closer to the shoreline could increase the height and severity of the waves, whereas a limited wave approach could reduce the wave height). Provided the projected wave action is not significantly different from that observed in the model, sizing of the revetment for other portions of the west slope can be accomplished by inserting the stability coefficient, calculated from acceptable model results, and the new design wave height into Hudson's stability equation.

Table 1

Hydrograph A, North Slope

Plans N-1, N-2, N-3, N-3-A, N-4, N-5, and N-6

Step	Still-Water Level ft msl	Test Wave Height* ft	Test Wave Period sec	Prototype Test Time hr	Wave Type
Shakedown	0.0	10.0	15	0.25	Shakedown
1	0.0	20.0	15	2.00	Worst breaking
2	0.0	22.7	17	0.50	Worst breaking
3	+1.9	21.2	15	2.00	Worst breaking
4	+1.9	23.3	17	0.50	Worst breaking

* Wave heights were measured during calibration at the -21.0 ft contour.

Table 2

Hydrograph B, West Slope

Plans W-1, W-2, W-3, and W-4

Step	Still-Water Level ft msl	Test Wave Height* ft	Test Wave Period sec	Prototype Test Time hr	Wave Type
Shakedown	0.0	2.5	9	0.25	Shakedown
1	0.0	6.25	9	1.00	Worst breaking
2	0.0	9.0	15	1.00	Worst breaking
3	0.0	8.5	17	0.50	Worst breaking
4	+1.9	8.9	9	1.00	Worst breaking
5	+1.9	9.6	15	1.00	Worst breaking
6	+1.9	10.5	17	0.50	Worst breaking

* Wave heights were measured during calibration at the -13.0 ft contour.

Table 3

Hydrograph 3D-A

Plan 3D-1

Step	Still-Water Level ft msl	Test Wave Height* ft	Test Wave Period sec	Prototype Test Time hr	Wave Type
Shakedown	0.0	12.0	15	0.25	Shakedown
1	0.0	23.5	15	2.00	Worst breaking
2	0.0	27.0	17	0.50	Worst breaking
3	+1.9	26.0	15	2.00	Worst breaking
4	+1.9	28.0	17	0.50	Worst breaking

* Wave heights were measured during calibration at the -25.0 ft contour.

Table 4

Hydrograph 3D-B

Plan 3D-1

Step	Still-Water Level ft msl	Test Wave Height* ft	Test Wave Period sec	Prototype Test Time hr	Wave Type
Shakedown	0.0	9.0	9	0.25	Shakedown
1	0.0	18.2	9	1.50	Breaking
2	0.0	19.4	13	1.50	Breaking
3	+1.9	19.6	9	1.50	Breaking
4	+1.9	20.9	13	1.50	Breaking

* Wave heights were measured during calibration at the -25.0 ft contour.

Table 5

Runup, North Slope, Plan N-2

Wave Period, sec	Wave Height, ft*	Runup,** ft above msl
	0.0 Still-Water Level	
7	5.0	2-3
7	10.0	3-4
7	13.0	6-7
9	5.0	2-3
9	10.0	5
9	15.0	7-8
11	5.0	2-3
11	10.0	6-8
11	15.0	9-9.5
11	17.0	9-10
13	5.0	2-3
13	10.0	5-6
13	15.0	8-9
13	18.0	12-13
15	5.0	2-3
15	10.0	5.5-6
15	15.0	7-8
15	20.0†	11-12
17	5.0	2-3
17	10.0	6-7
17	15.0	8-8.5
17	20.0	13-14
17	22.0	17-18
17	22.7†	19-20
	+1.9 Still-Water Level	
7	5.0	3-4
7	10.0	6-7
7	15.0	8-8.5
9	5.0	3-5
9	10.0	7-8.5
9	15.0	10-11
11	5.0	5-6
11	10.0	8-9

(Continued)

* Incident wave heights were measured during calibration at the -21.0 ft contour.

** Runup measured on roughened slope with culvert open.

† Hydrograph A test conditions.

Table 5 (Concluded)

Wave Period, sec	Wave Height, ft	Runup, ft above msl
+1.9 Still-Water Level (Continued)		
11	15.0	13-14
11	19.0	15-16
13	5.0	5.5-6.5
13	10.0	9-10
13	15.0	11-12.5
13	21.0	19-20
15	5.0	3-5
15	10.0	8-9
15	15.0	10-12
15	21.2†	20-21
17	5.0	6
17	10.0	8-10
17	15.0	13-14
17	20.0	20-21
17	23.3†	24-24.5
17	24.0	25-26

† Hydrograph A test conditions.

Table 6

Runup, Northslope, Plan N-3-A

+1.9 ft msl Still-Water Level

Wave Period, sec	Wave Height, ft*	Runup,** ft above msl
7	5.0	3-4
7	10.0	6-7
7	15.0	8-8.5
9	5.0	4-5
9	10.0	7-8.5
9	15.0	10-11
11	5.0	5.5-6
11	10.0	9-10
11	15.0	11-12
11	19.0	14-15
13	5.0	6-6.5
13	10.0	9-9.5
13	15.0	11-12.5
13	21.0	18-19
15	5.0	4-5
15	10.0	8-8.5
15	15.0	11-12
15	21.2†	20-21
17	5.0	5-6
17	10.0	9-10
17	15.0	13-14
17	20.0	19-20
17	23.3†	23-25
17	24.0	22-23

* Incident wave heights were measured during calibration at the -21.0 ft contour.

** Runup measured on roughened slope with culvert open.

† Hydrograph A test conditions.

Table 7

Wave and swl Conditions Tested with Trestle and Plan N-3-A with Trestle

Wave Period, sec	Wave Height,* ft
+1.9 Still-Water Level	
7.0	5.0
7.0	10.0
7.0	15.0
8.0	10.0
8.0	12.0
8.0	15.0
9.0	5.0
9.0	10.0
9.0	15.0
9.0	18.0
11.0	5.0
11.0	10.0
11.0	12.0
11.0	15.0
11.0	19.0
13.0	5.0
13.0	12.0
13.0	15.0
13.0	17.0
15.0	5.0
15.0	12.0
15.0	15.0
15.0	17.0
17.0	5.0
17.0	12.0
17.0	15.0
17.0	17.0
17.0	20.0
0.0 Still-Water Level	
8.0	10.0
8.0	12.0
8.5	12.0
8.5	15.0

(Continued)

* Incident wave height measured during calibration at the -21.0 contour.

Table 7 (Concluded)

Wave Period, sec	Wave Height, ft
0.0 Still-Water Level (Continued)	
9.0	10.0
9.0	15.0
11.0	15.0
15.0	17.0
15.0	20.0
17.0	17.0
17.0	20.0

Table 8

Runup, North Slope, Plan N-6

Wave Period, sec	Wave Height,* ft	Runup,** ft above msl
	0.0 Still-Water Level	
7	5.0	3-4
7	10.0	4-5
7	13.0	5-6
9	5.0	3-4
9	10.0	5-6
9	15.0	6-7
11	5.0	2-3
11	10.0	5-6
11	15.0	8-9
11	17.0	8-9
13	5.0	2-3
13	10.0	6-6.5
13	15.0	7-8
13	18.0	9-10
15	5.0	2-3
15	10.0	5-6
15	15.0	6-7
15	20.0†	9-10
17	5.0	2-3
17	10.0	5-6
17	15.0	8-9
17	20.0	12-13
17	22.0	15-16
17	22.7†	15-16
	+1.9 Still-Water Level	
7	5.0	4-5
7	10.0	6-7
7	15.0	8-9
9	5.0	3-5
9	10.0	7-8
9	15.0	10-10.5
11	5.0	4.5-5.5
11	10.0	7.5-8.5

(Continued)

* Incident wave heights measured during calibration at the -21.0 ft msl contour.

** Runup measured on roughened slope with culvert open.

† Hydrograph A test conditions.

Table 8 (Concluded)

Wave Period, sec	Wave Height, ft	Runup, ft above msl
+1.9 Still-Water Level (Continued)		
11	15.0	8-9
11	19.0	13-14
13	5.0	5-6
13	10.0	8-9.5
13	15.0	11-12
13	21.0	16-17
15	5.0	4-5.5
15	10.0	8-8.5
15	15.0	10-11
15	21.2†	17-18
17	5.0	5-6.5
17	10.0	9-10
17	15.0	13-14
17	20.0	17-18
17	23.3†	21-22
17	24.0	22-23

† Hydrograph A test conditions.

Table 9

Runup, West Slope, Plan W-2

+1.9 Still-Water Level

Wave Period, sec	Wave Height,* ft	Runup, ft above msl
7	3.2	5-6
7	6.9	11-13
7	7.7	11-13
7	11.2	9-11
9	2.6	3-5
9	5.5	9-10
9	6.8	11-13
9	8.9	13-15
11	4.0	7-9
11	8.2	11-13
11	9.7	13-14
13	4.8	9-10
13	8.7	15-17
13	9.7	16-17
13	11.4	15-16
15	4.5	5-7
15	9.6	11-13
15	9.9	12-16
15	10.6	15-17
17	3.8	7-9
17	8.9	13-15
17	9.9	14-15
17	10.5	15-16

* Incident wave heights measured during calibration at the -13.0 ft contour.

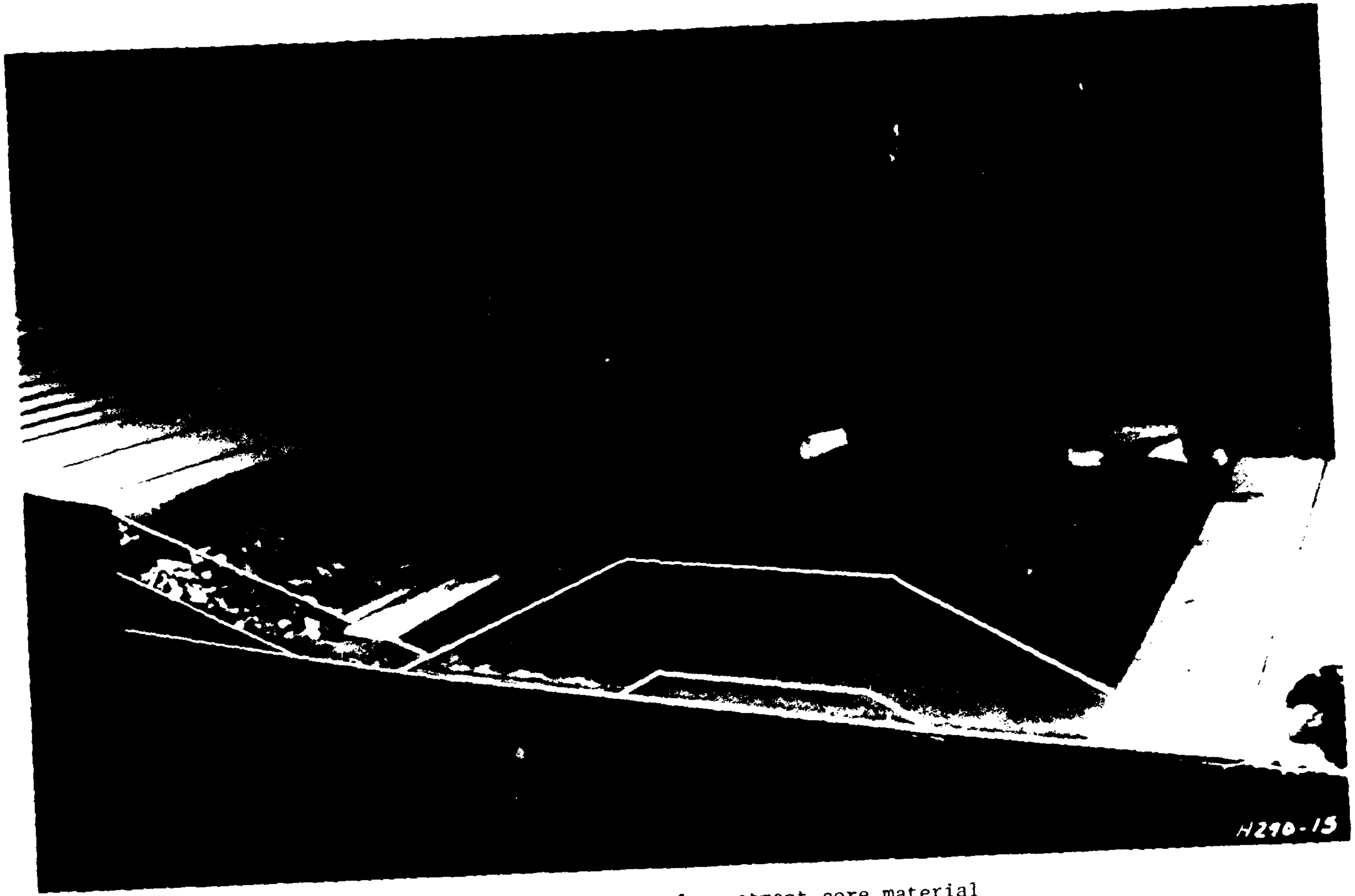


Photo 1. Placement of revetment core material

Photo 2. Adjusting core material to correct elevation and slope

Photo 3. Placement of revetment armor stone

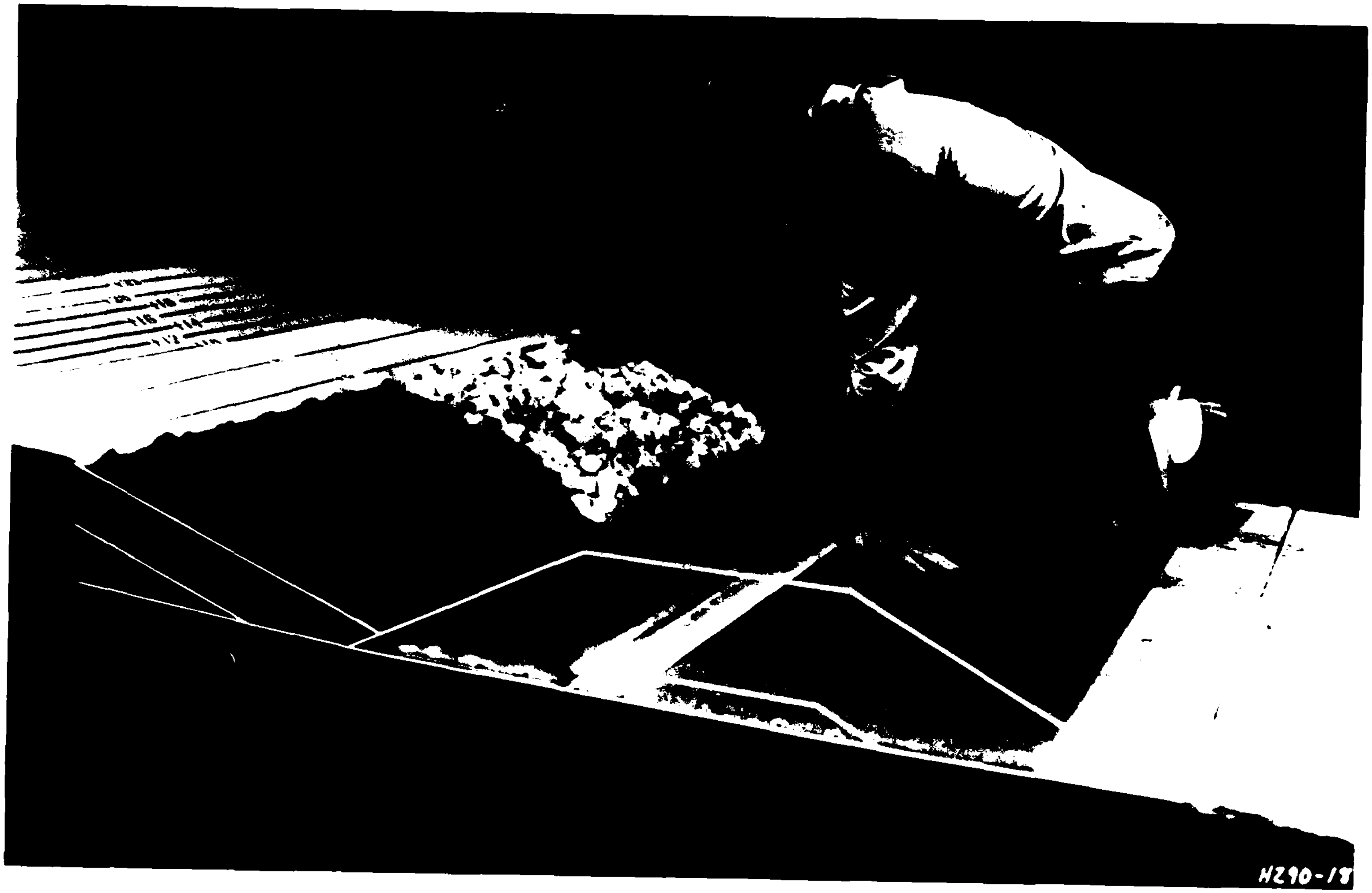


Photo 4. Completed revetment section

Photo 5. Construction of the breakwater's beach-side toe

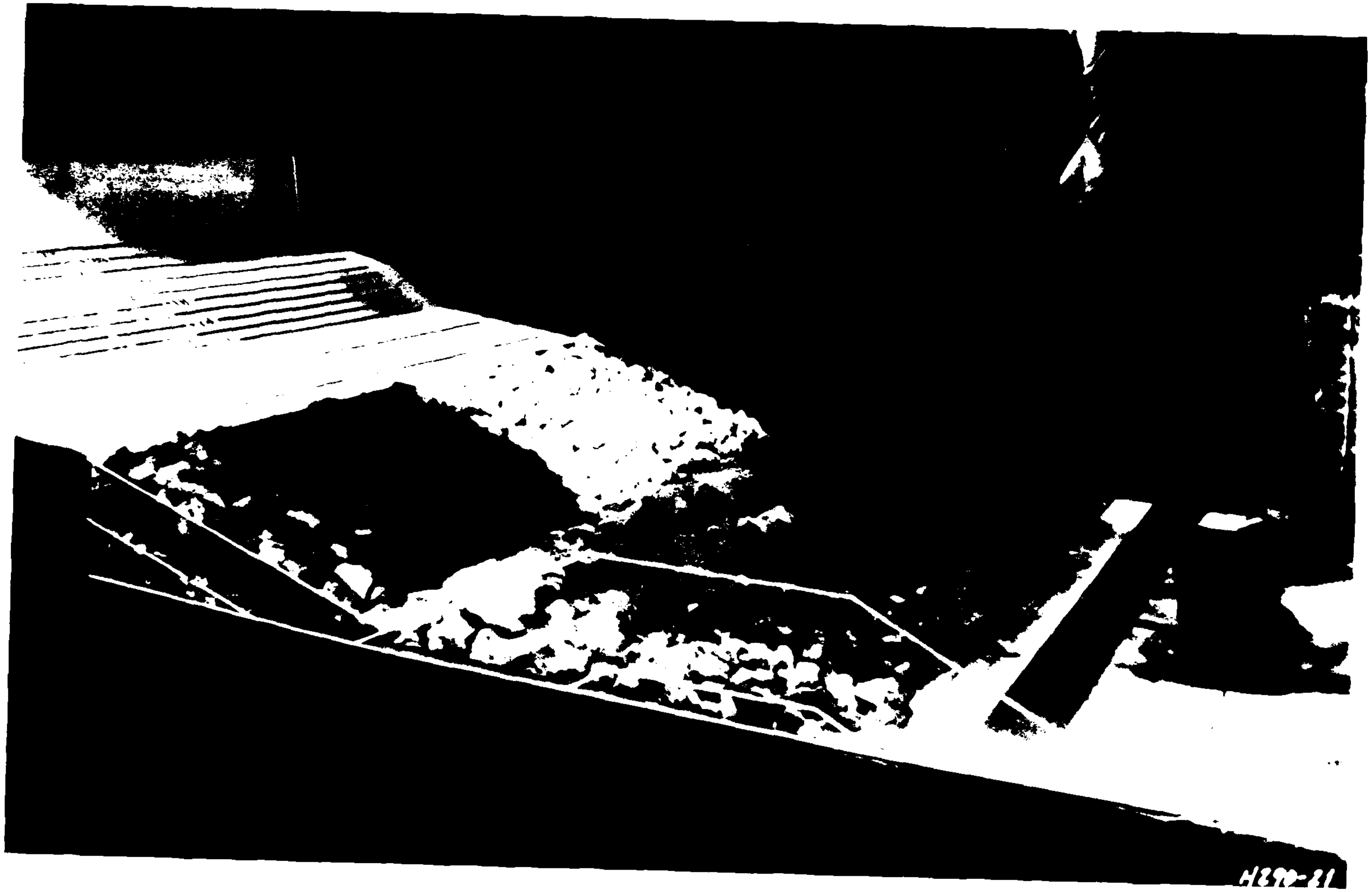


Photo 6. Placement of breakwater core material

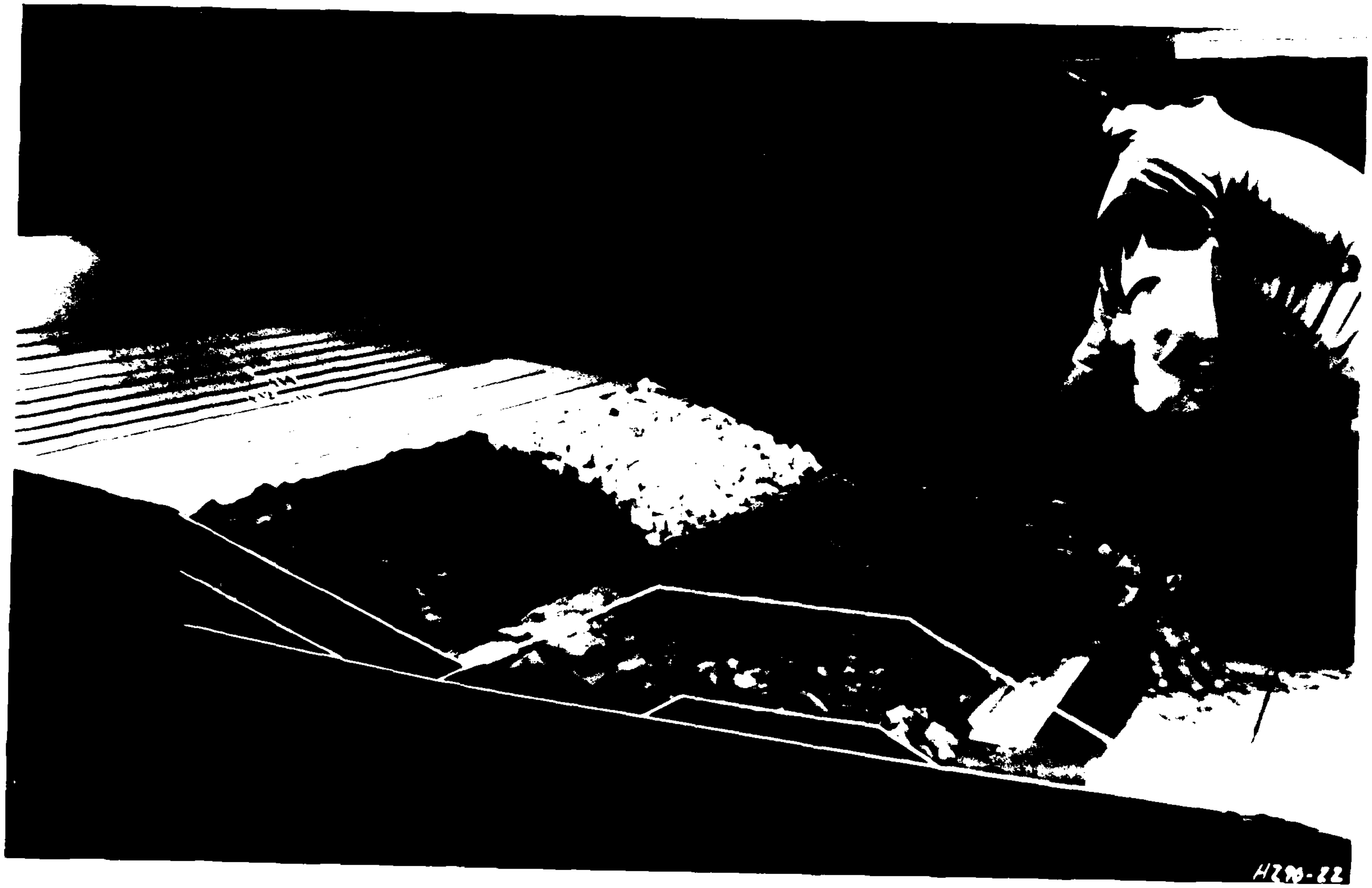


Photo 7. Adjusting core material to correct elevation and slope

Photo 8. Breakwater armor-stone placement

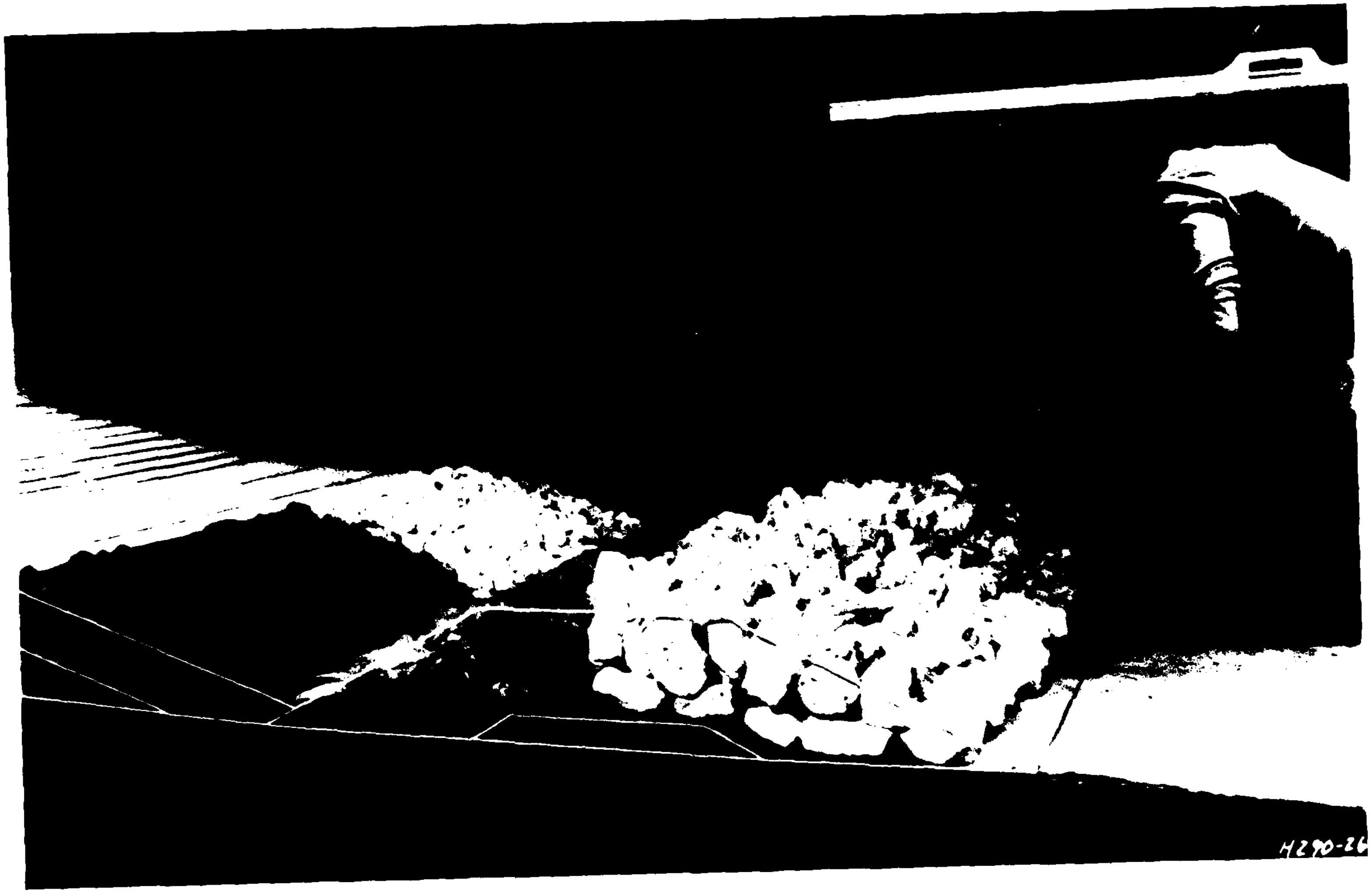


Photo 9. Completed offsore breakwater and revetment test section

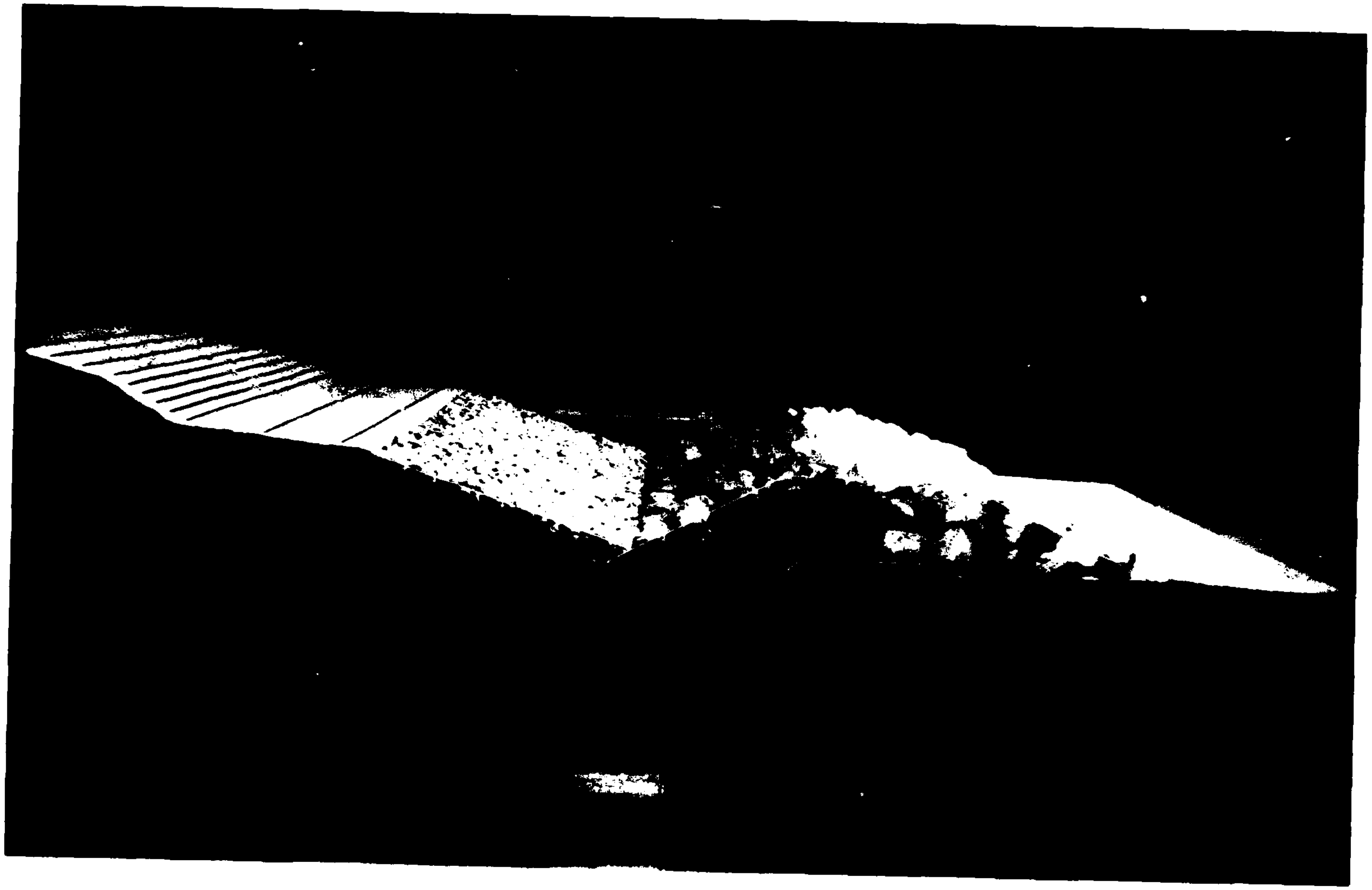


Photo 10. Side view of Plan N-1 before testing, 1st test section

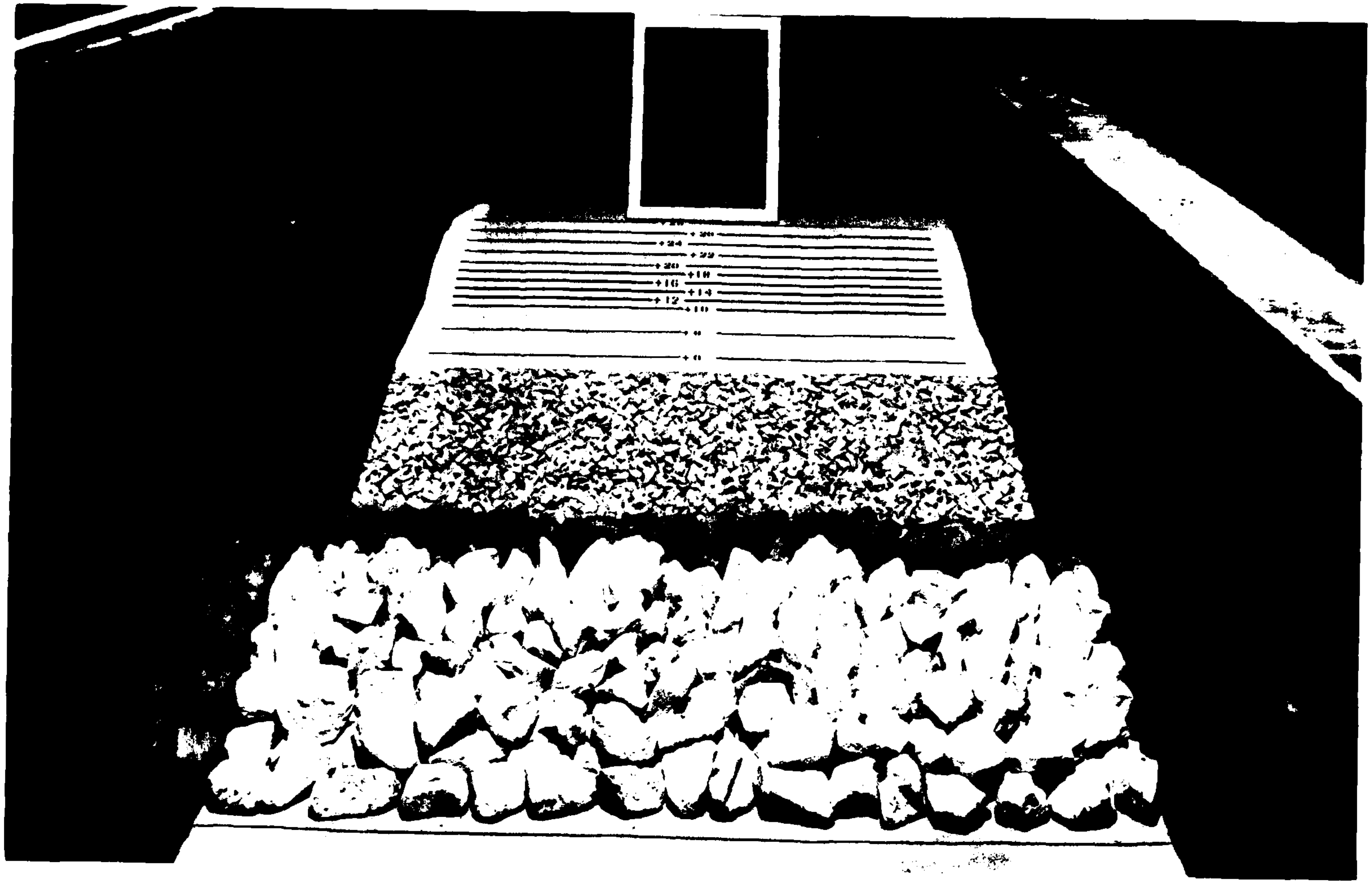


Photo 11. Sea-side view of Plan N-1 before testing, 1st test section

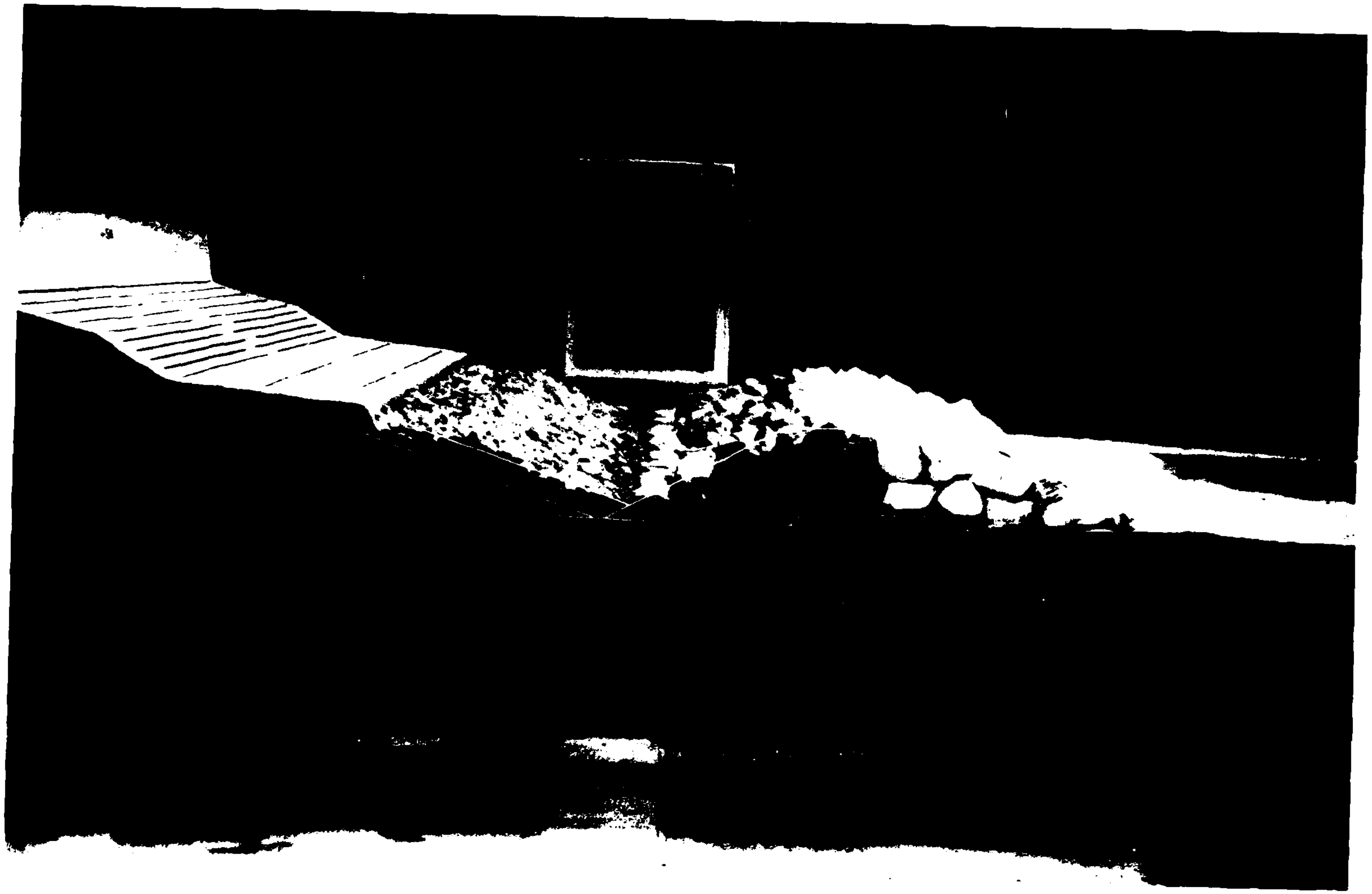


Photo 12. Side view of Plan N-1 after testing Hydrograph A (Plate 1 and Table 1), first test section

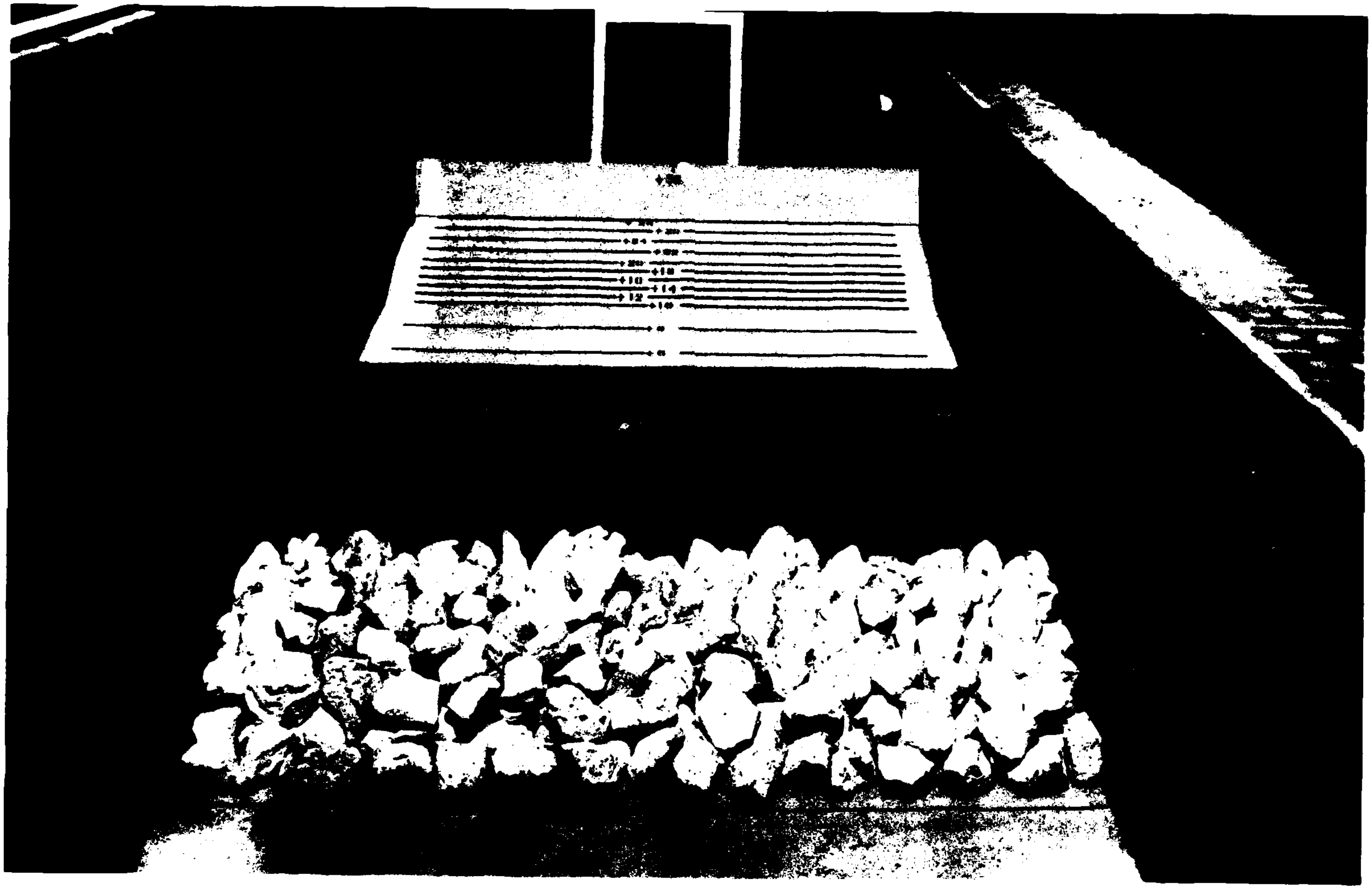


Photo 13. Sea-side view of Plan N-1 after testing Hydrograph A (Plate 1 and Table 1), 1st test section

Photo 14. Side view of Plan N-1 after testing Hydrograph A (Plate 1 and Table 1), 2nd test section

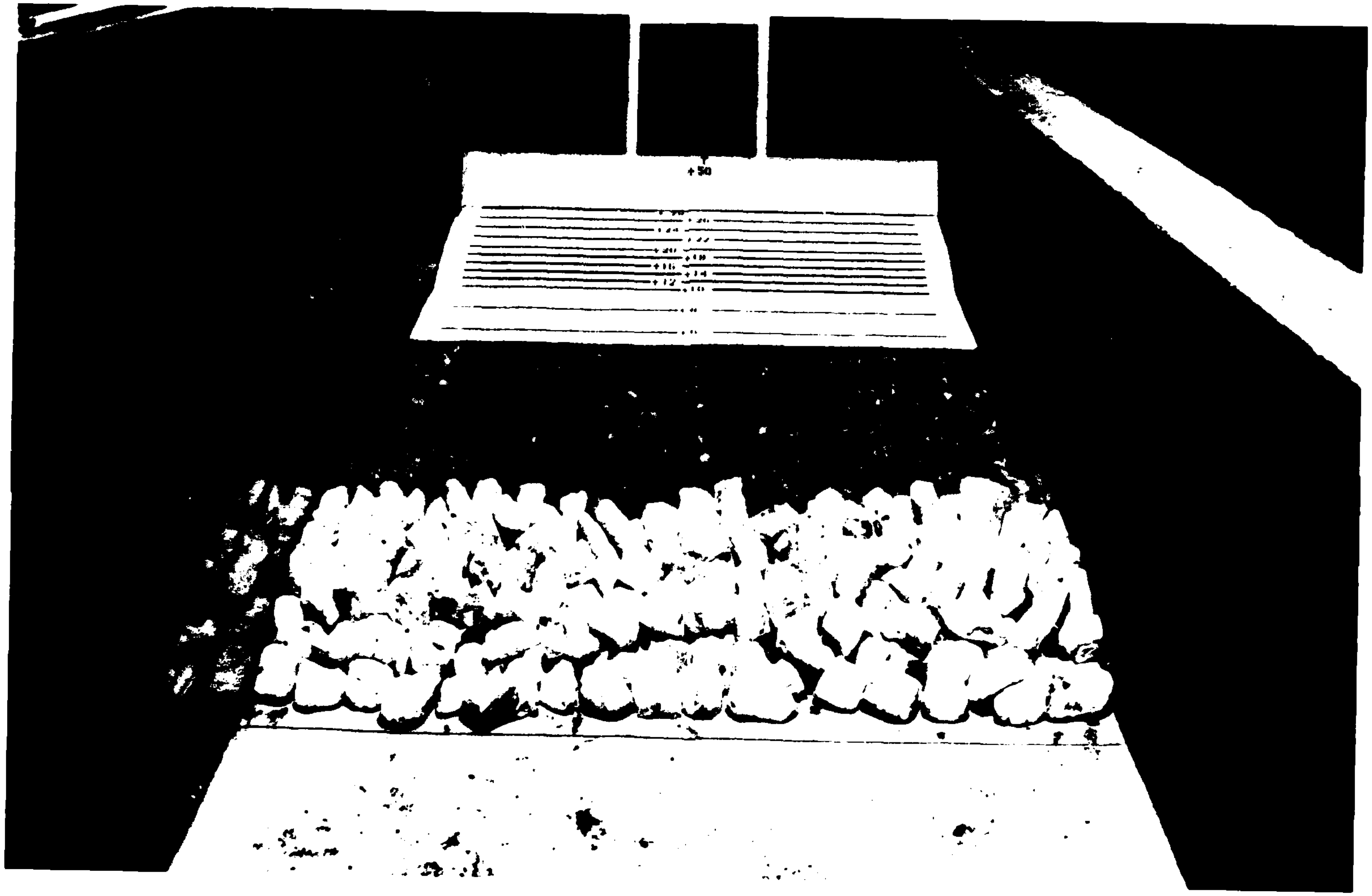


Figure 15. Sea-side view of Plan N-1 after testing Hydrograph A (Plate 1 and Table 1), 2nd test section

Photo 16. Side view of Plan N-2 before testing, 1st test section without culvert

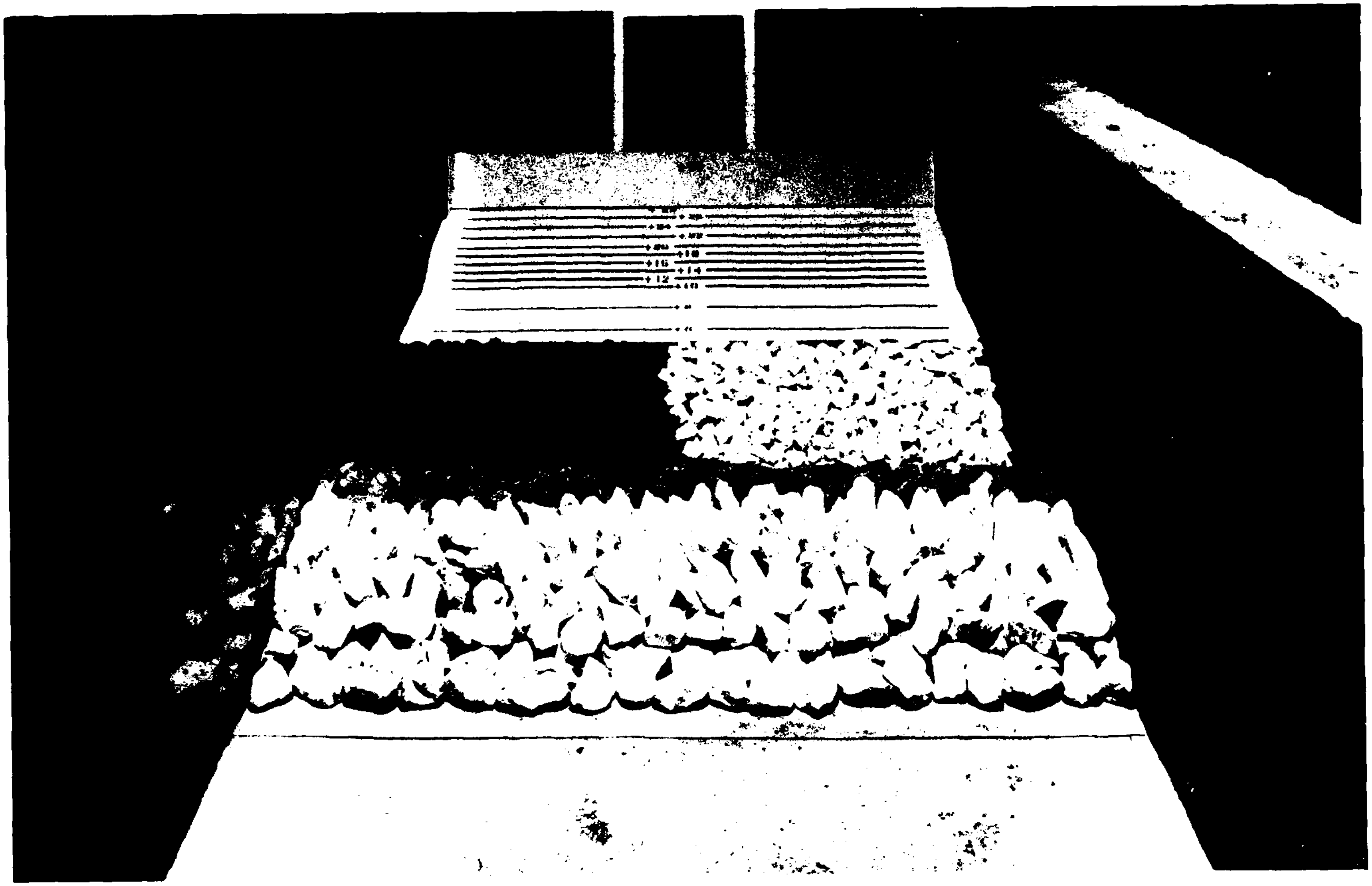


Photo 17. Sea-side view of Plan N-2 before testing, 1st test section without culvert

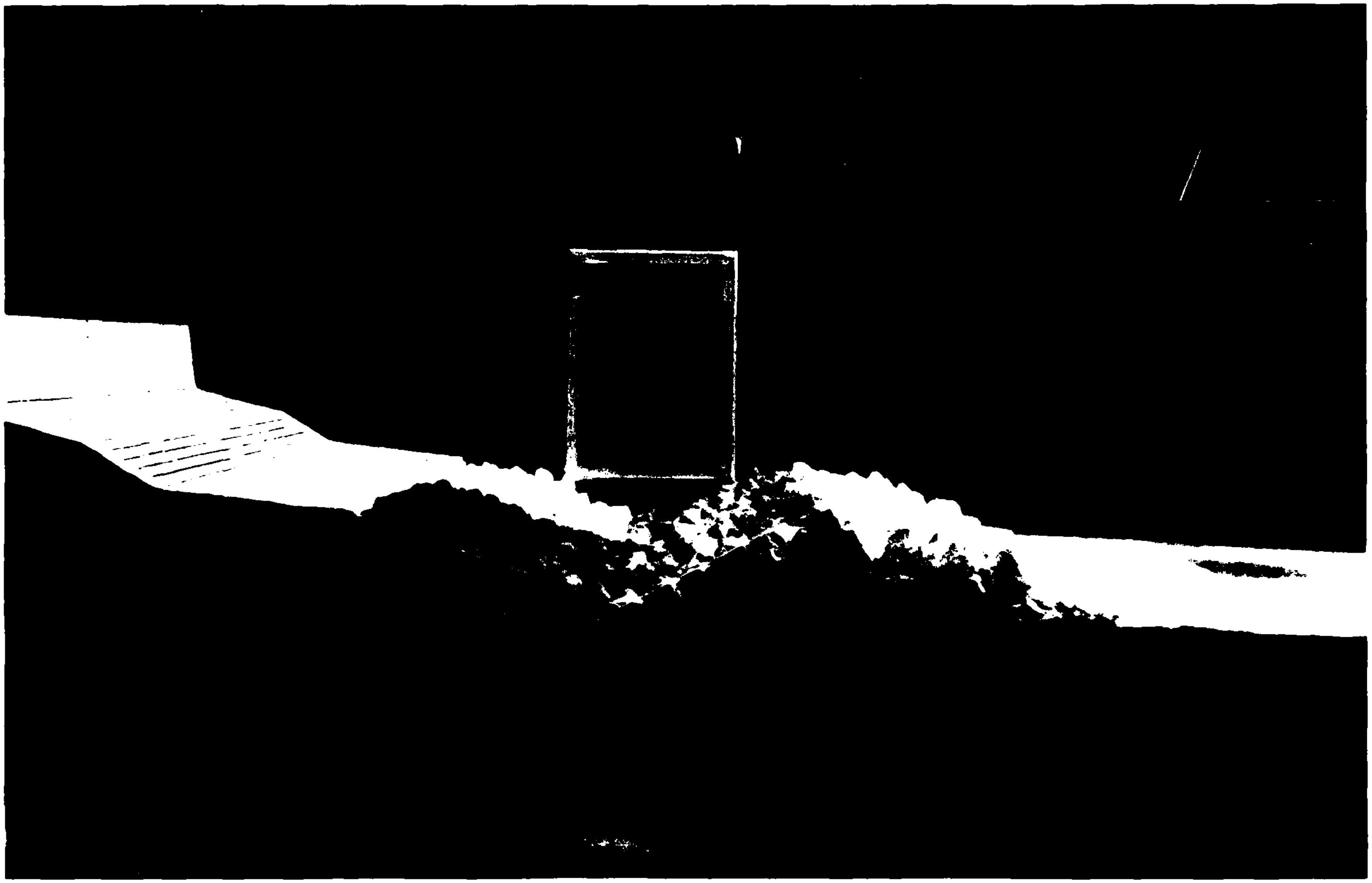


Photo 18. Side view of Plan N-2 after testing Hydrograph A (Plate 1 and Table 1), 1st test section without culvert

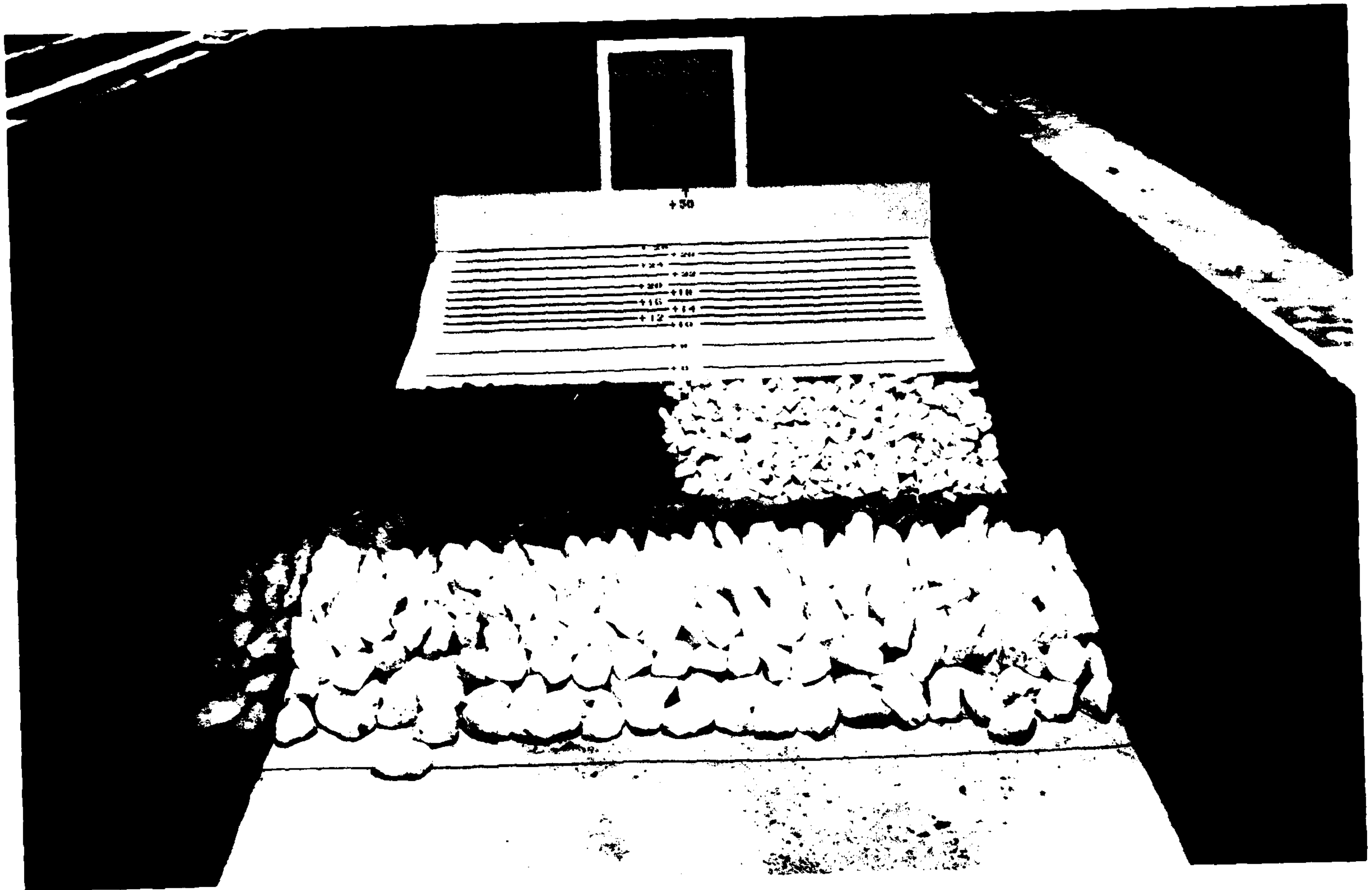


Photo 19. Sea-side view of Plan N-2 after testing Hydrograph A (Plate 1 and Table 1), 1st test section without culvert

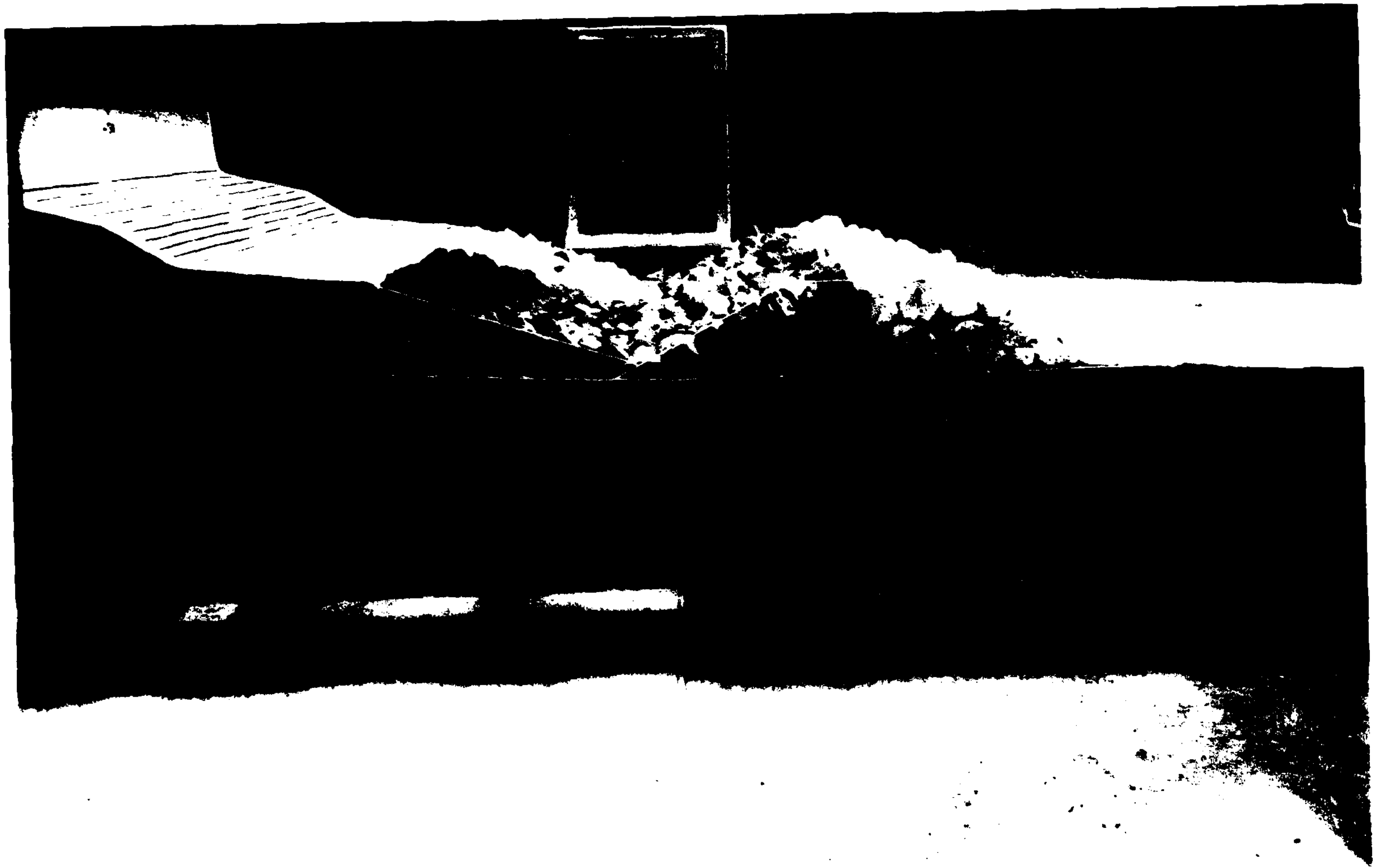


Photo 20. Side view of Plan N-2 after testing Hydrograph A (Plate 1 and Table 1), 2nd test section without culvert

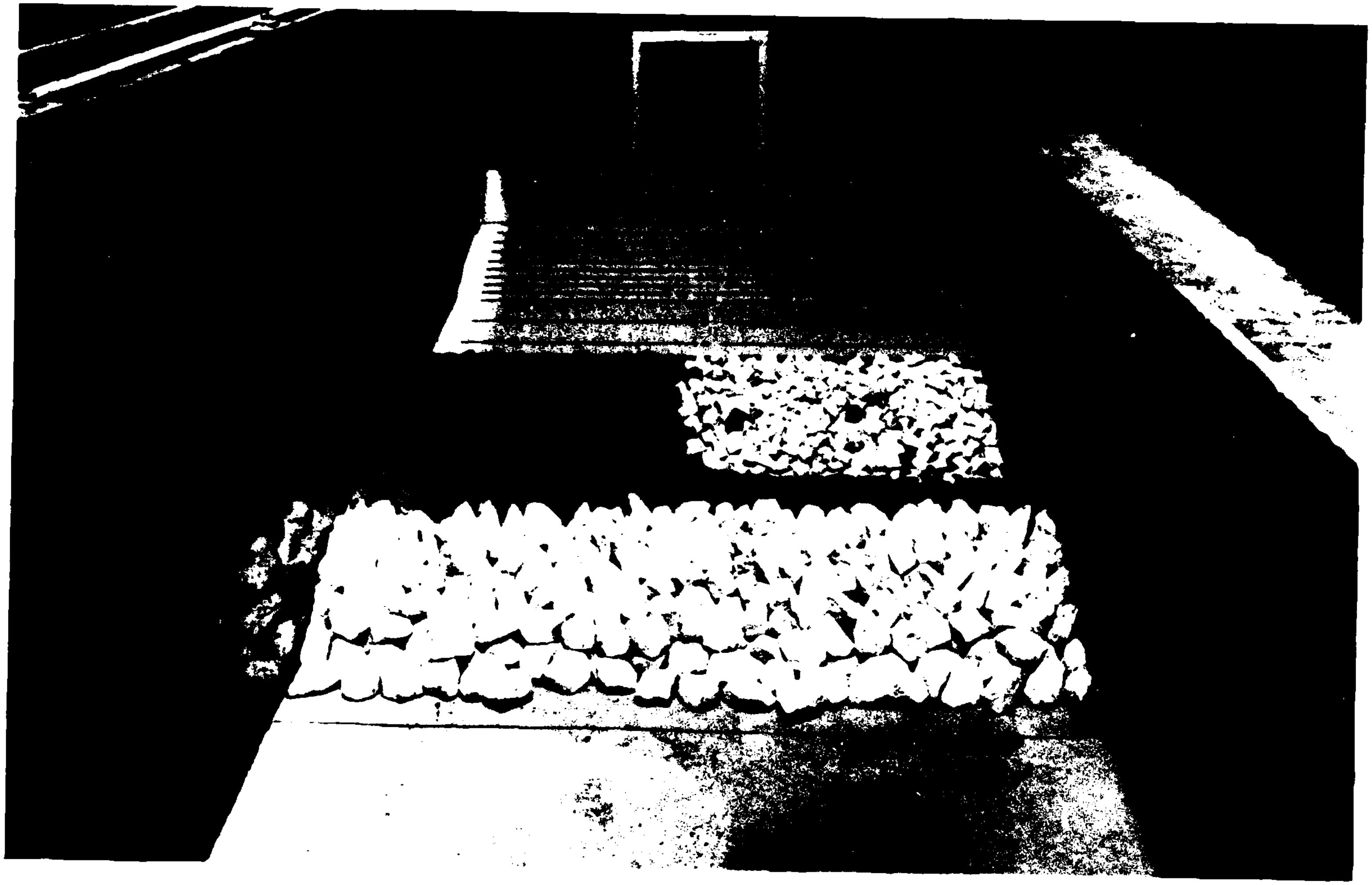


Photo 21. Sea-side view of Plan N-2 after testing Hydrograph A (Plate 1 and Table 1), 2nd test section without culvert

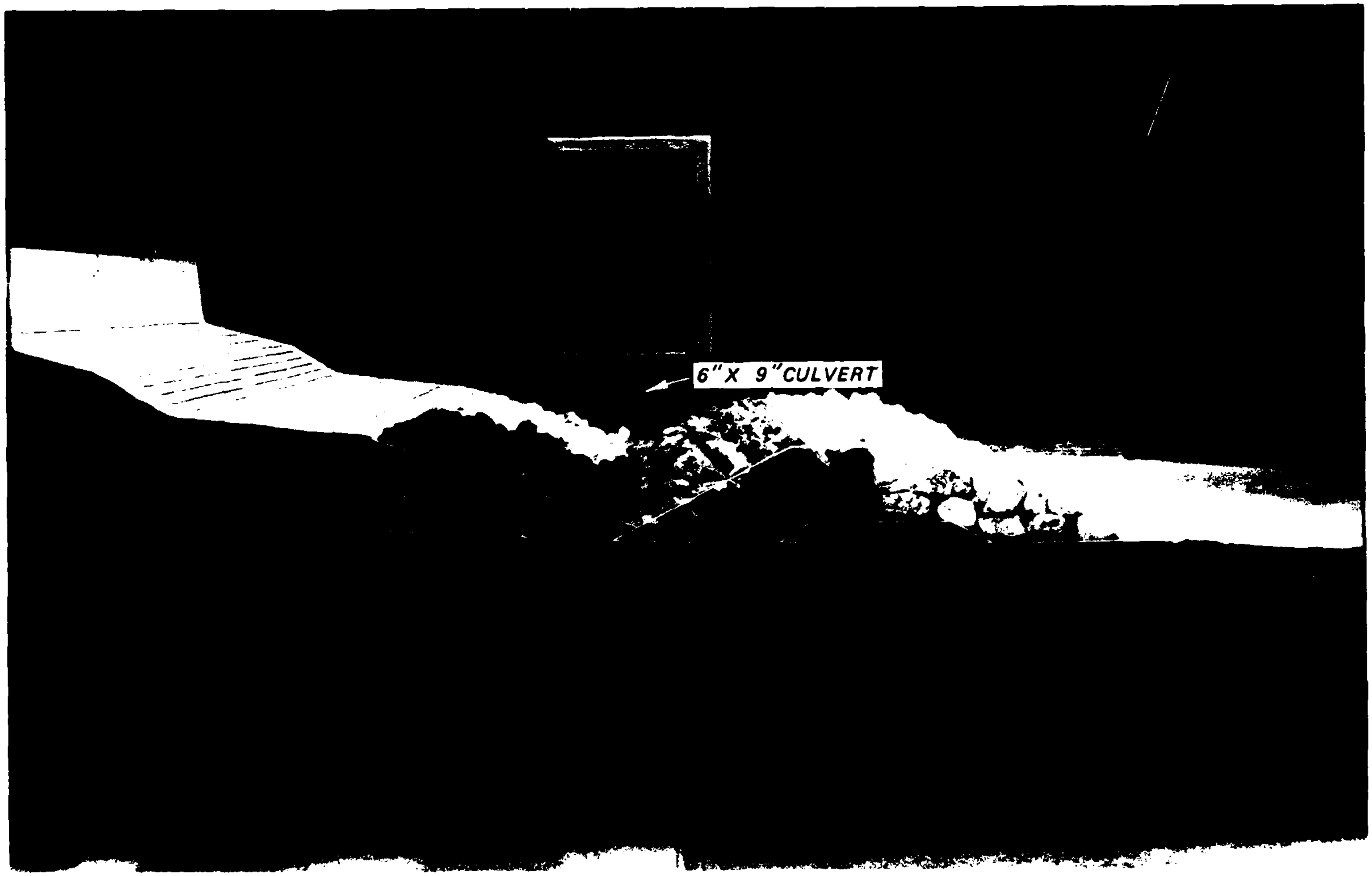


Photo 22. Side view of Plan N-2 after testing Hydrograph A (Plate 1 and Table 1), 1st test section with culvert

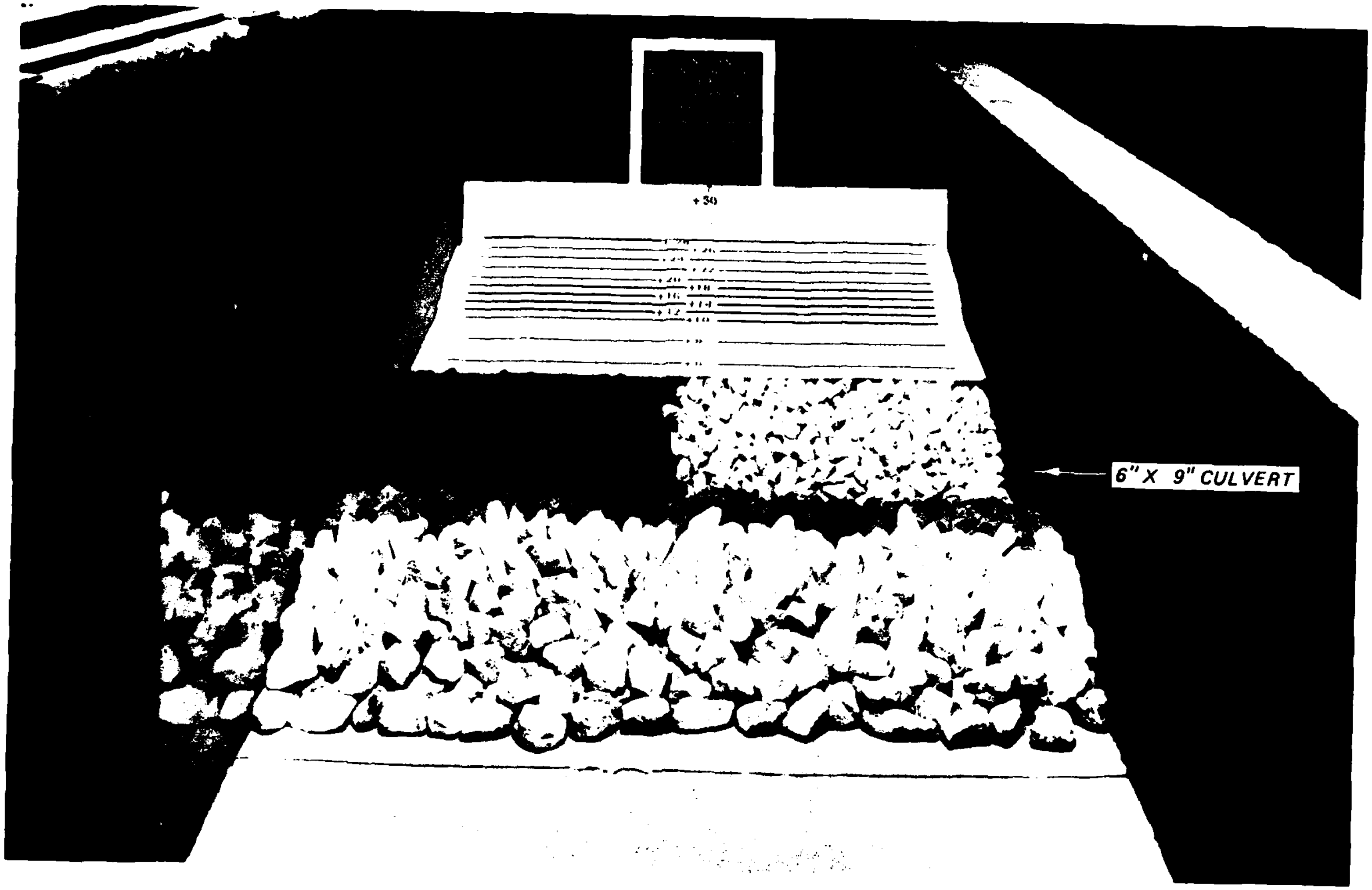


Photo 23. Sea-side view of Plan N-2 after testing Hydrograph A (Plate 1 and Table 1), 1st test section with culvert

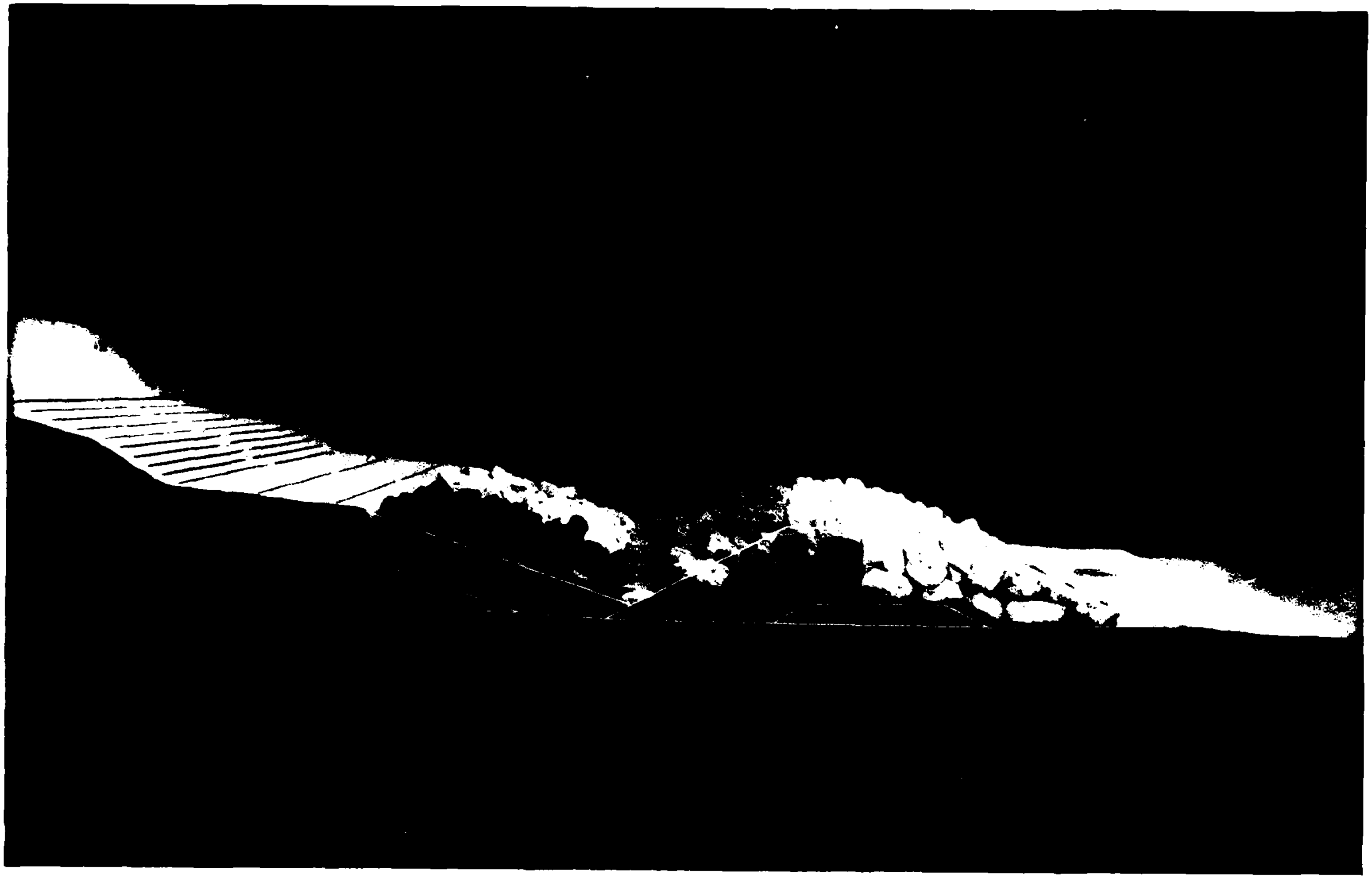


Photo 24. Side view of Plan N-2 after testing Hydrograph A (Plate 1 and Table 1), 2nd test section with culvert

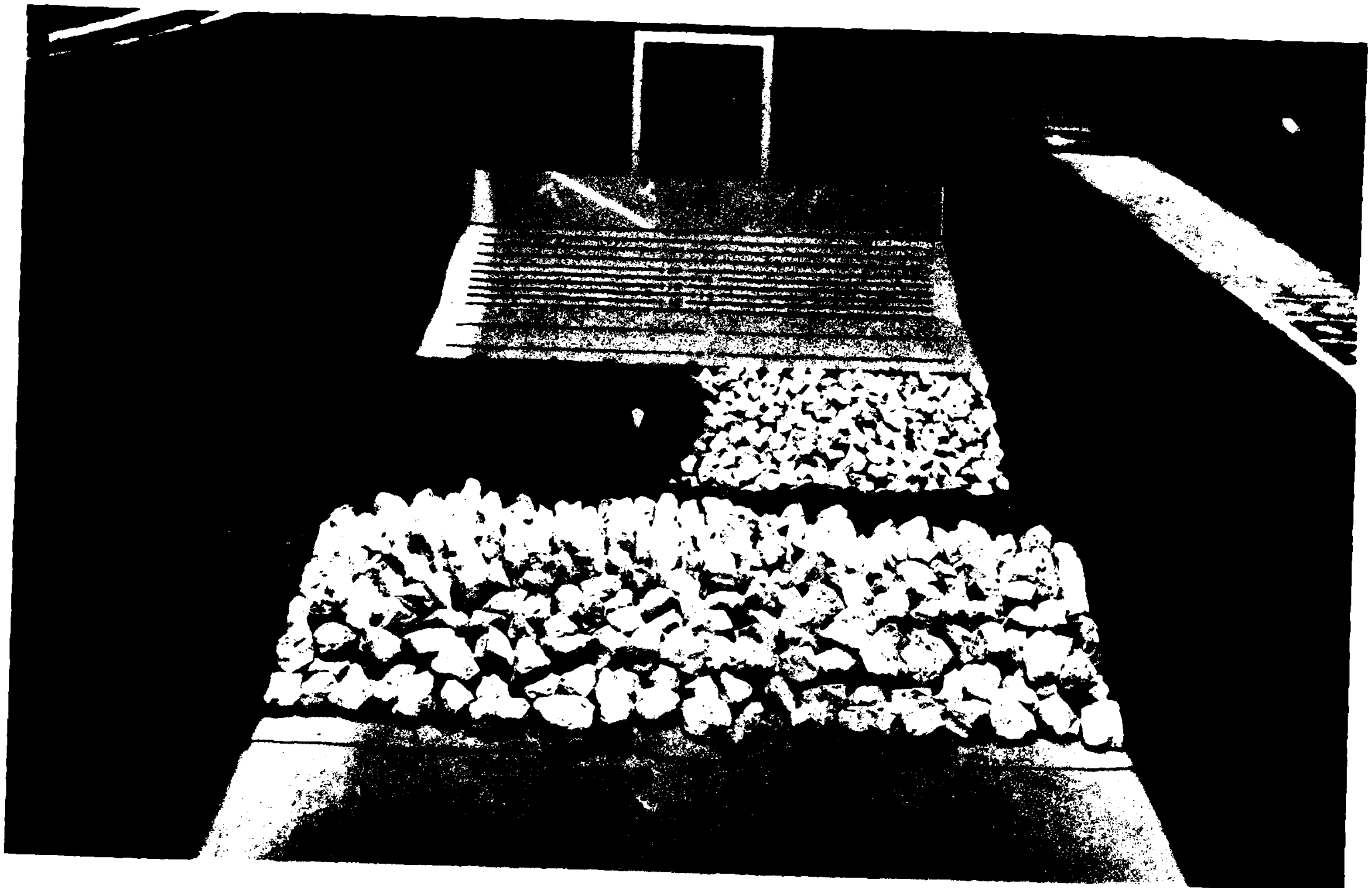


Photo 25. Sea-side view of Plan N-2 after testing Hydrograph A (Plate 1 and Table 1), 2nd test section with culvert

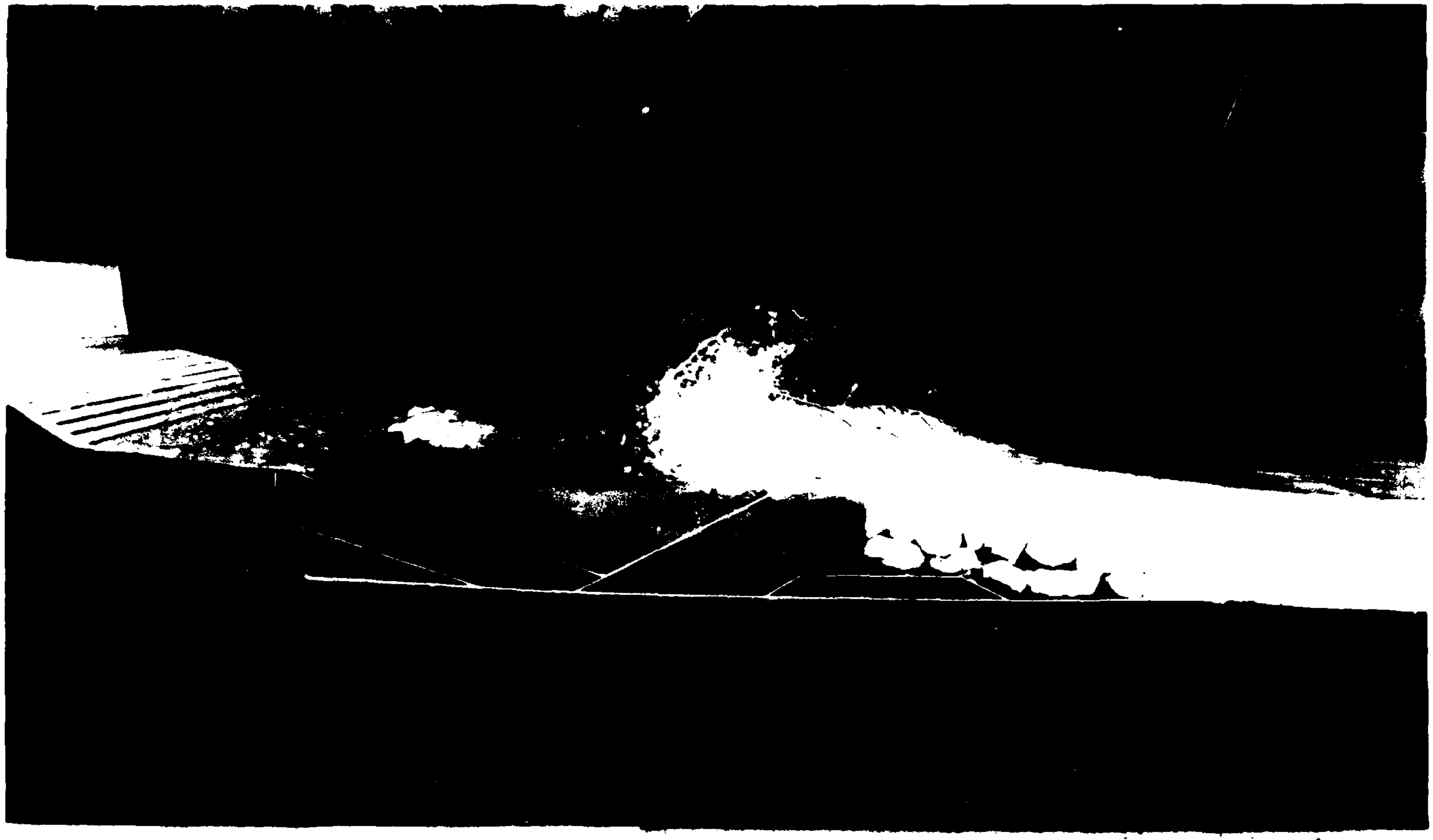


Photo 26. Side view of Plan N-2, 15.0-sec, 20.0-ft breaking wave at 0.0 swl, Step 1 of Hydrograph A (Plate 1 and Table 1)

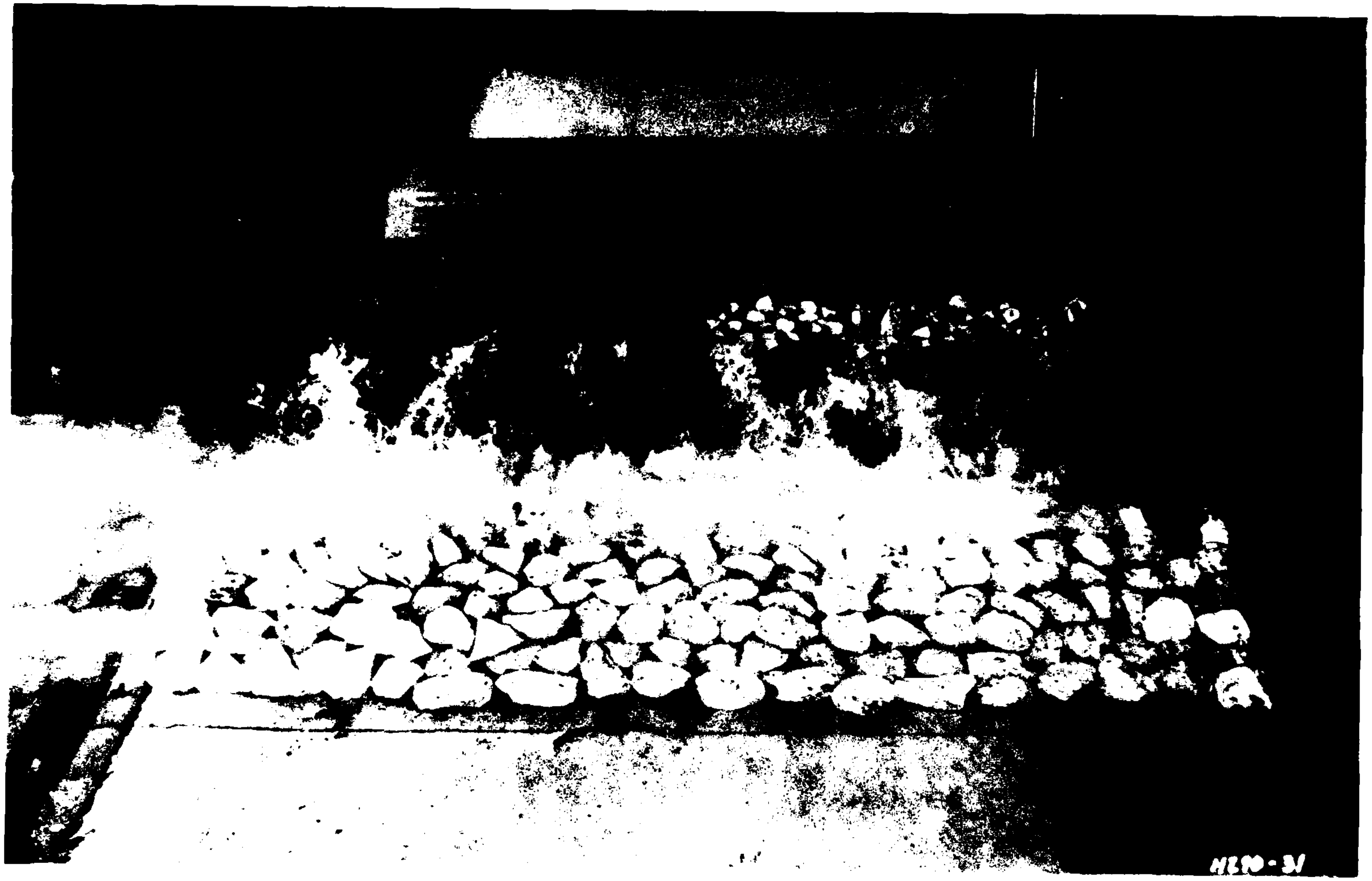


Photo 27. Sea-side view of Plan N-2, 15.0-sec, 20.0-ft breaking wave at 0.0 swl, Step 1 of Hydrograph A (Plate 1 and Table 1)

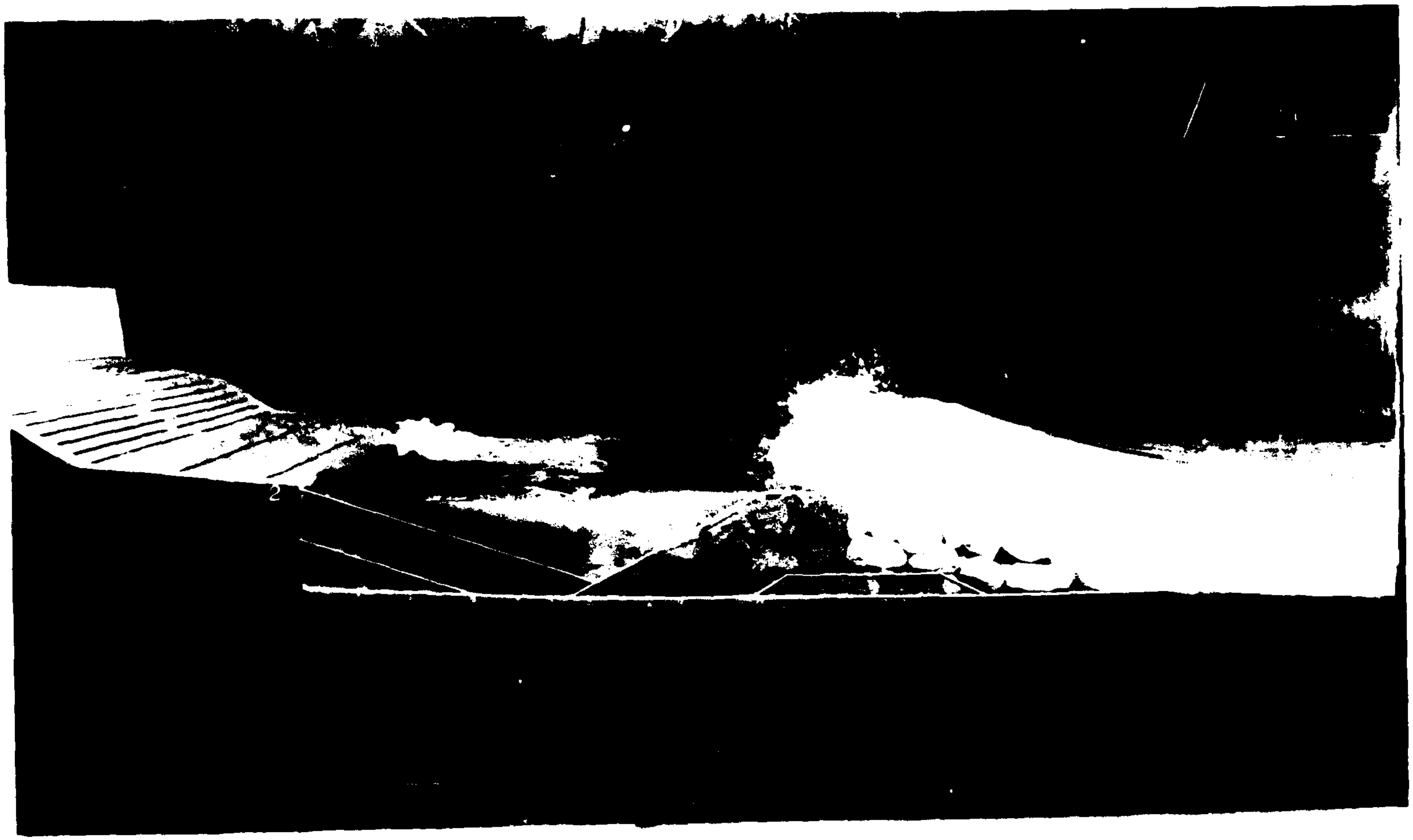


Photo 28. Side view of Plan N-2, 17.0-sec, 22.7-ft breaking wave at 0.0 swl, Step 2 of Hydrograph A (Plate 1 and Table 1)

Photo 29. Sea-side view of Plan N-2, 17.0-sec, 22.7-ft breaking wave at 0.0 swl, Step 2 of Hydrograph A (Plate 1 and Table 1)

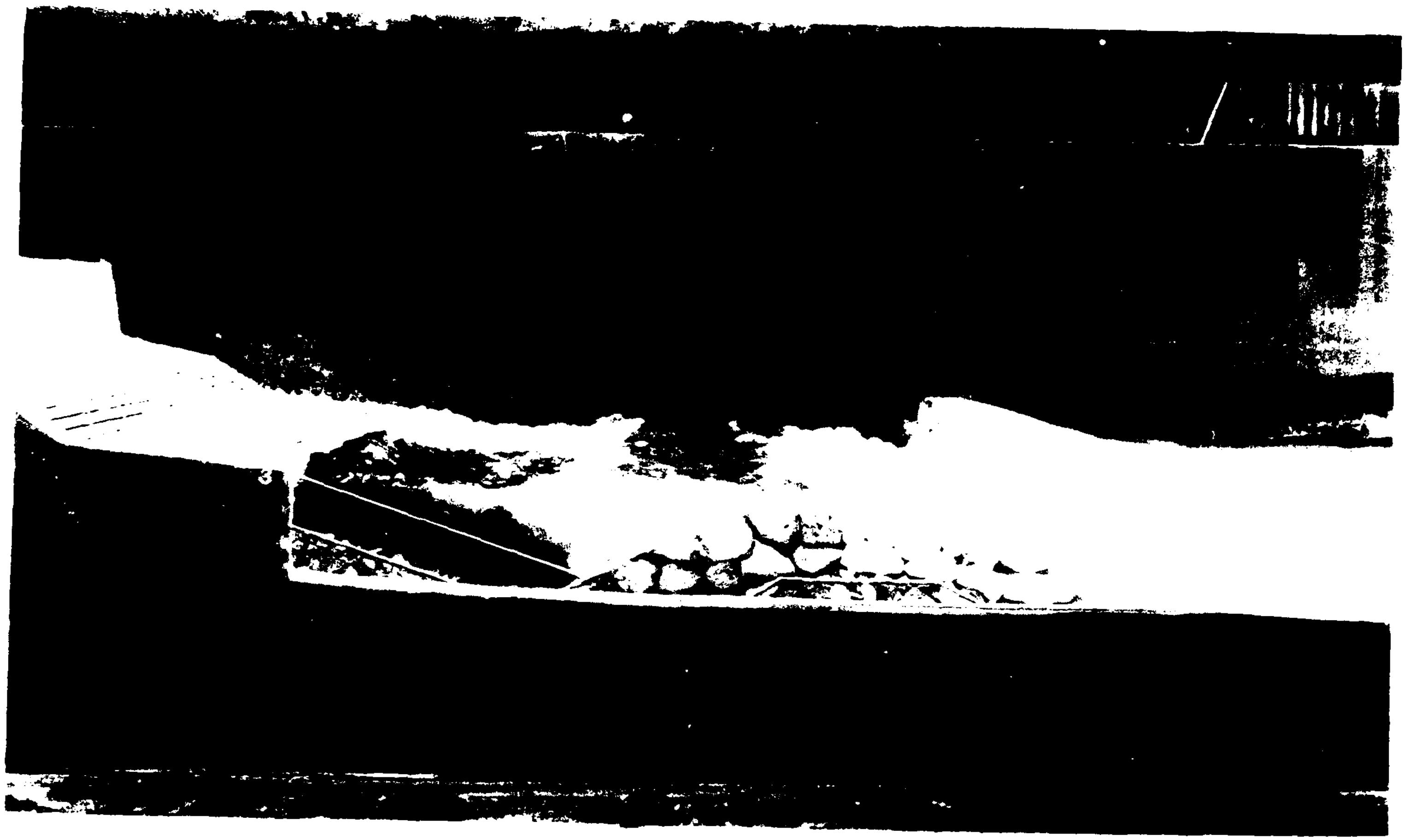


Photo 30. Side view of Plan N-2, 15.0-sec, 21.2-ft breaking wave at +1.9 swl, Step 3 of Hydrograph A (Plate 1 and Table 1)

Photo 31. Sea-side view of Plan N-2, 15.0-sec, 21.2-ft breaking wave at +1.9 swl, Step 3 of Hydrograph A (Plate 1 and Table 1)

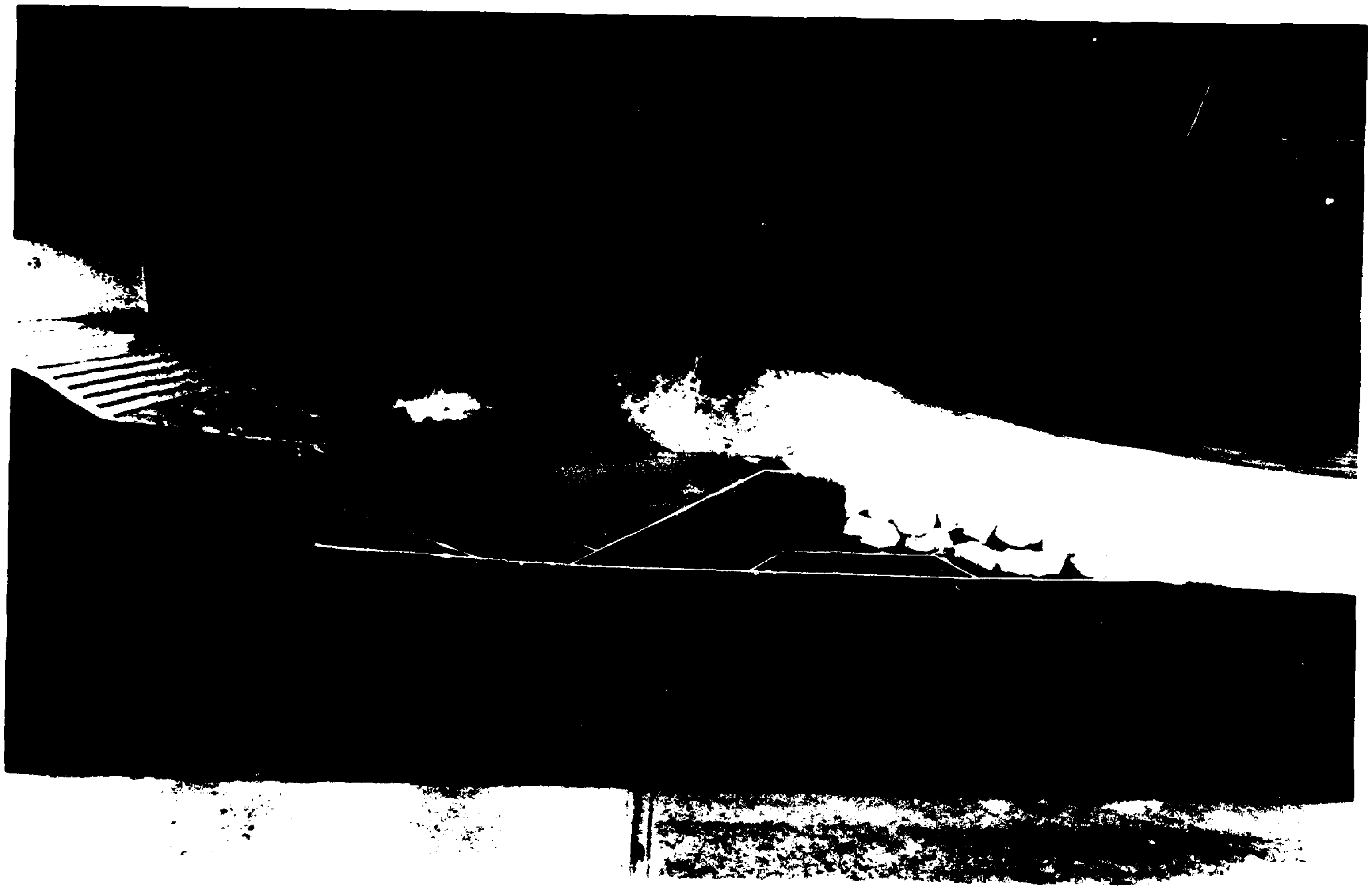


Photo 32. Side view of Plan N-2, 17.0-sec, 23.3-ft breaking wave at +1.9 swl, Step 4 of Hydrograph A (Plate 1 and Table 1)

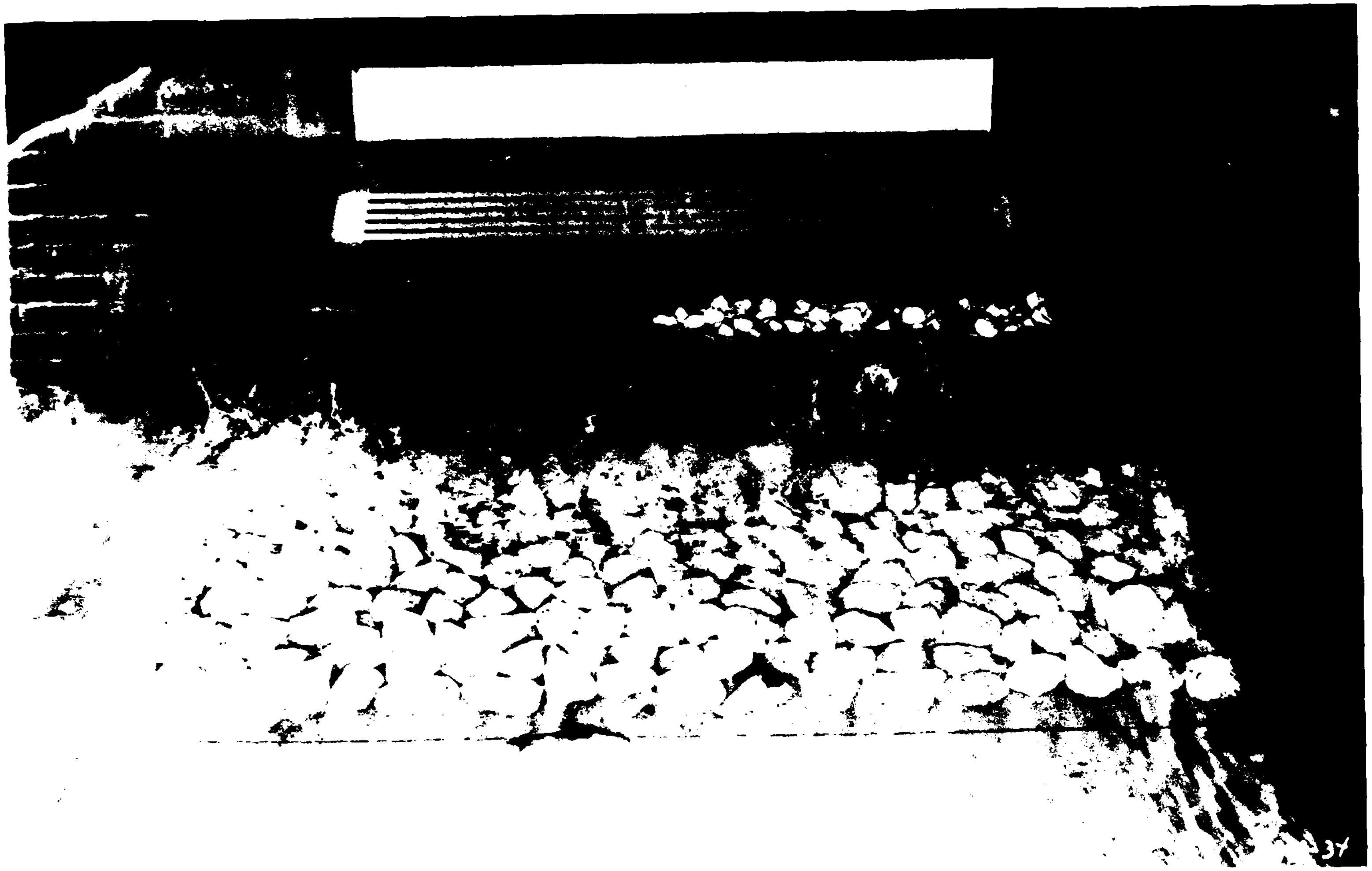


Photo 33. Sea-side view of Plan N-2, 17.0-sec, 23.3-ft breaking wave at +1.9 swl, Step 4 of Hydrograph A (Plate 1 and Table 1)

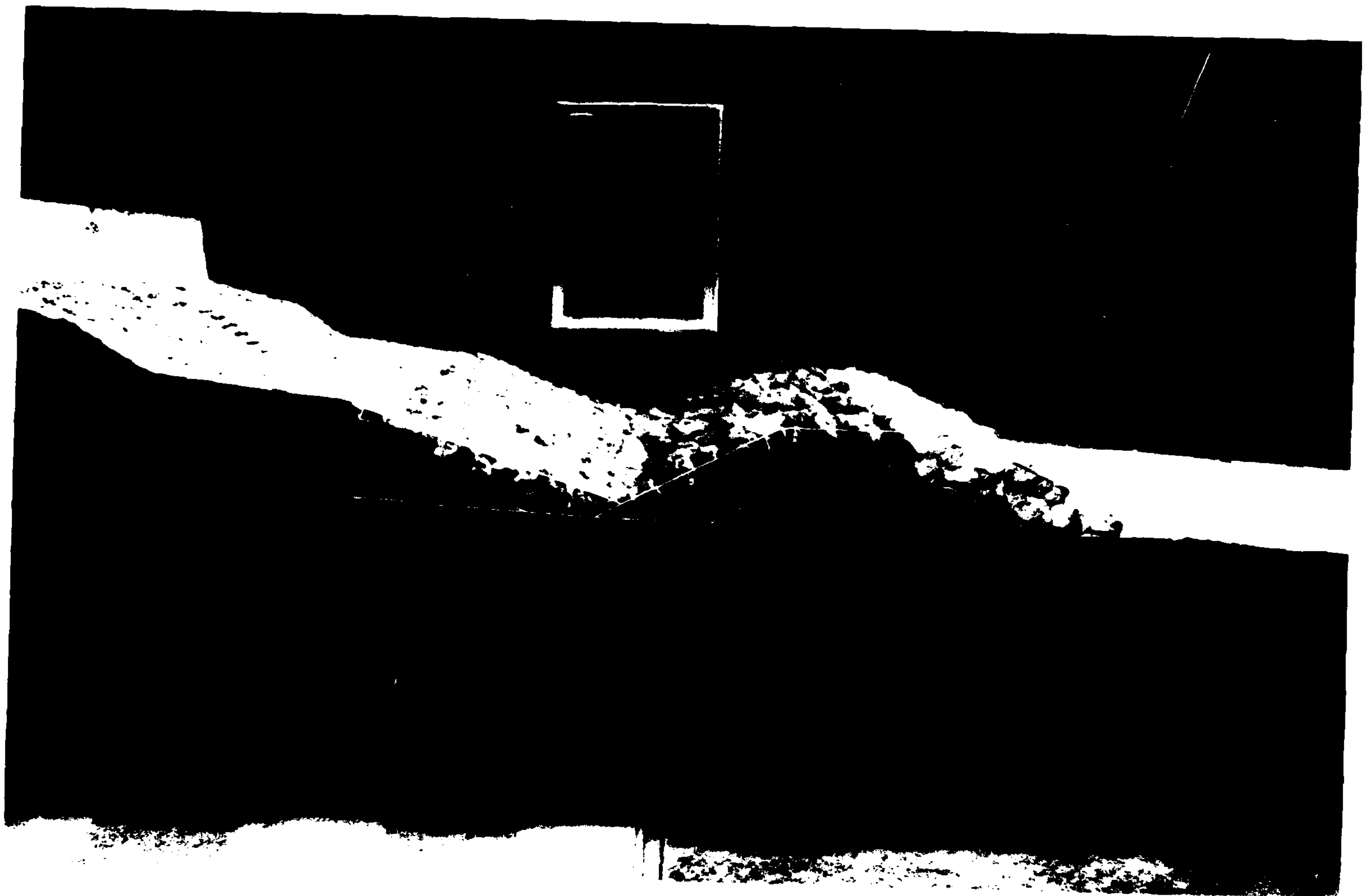


Photo 34. Side view of Plan N-3 before testing, 1st test section

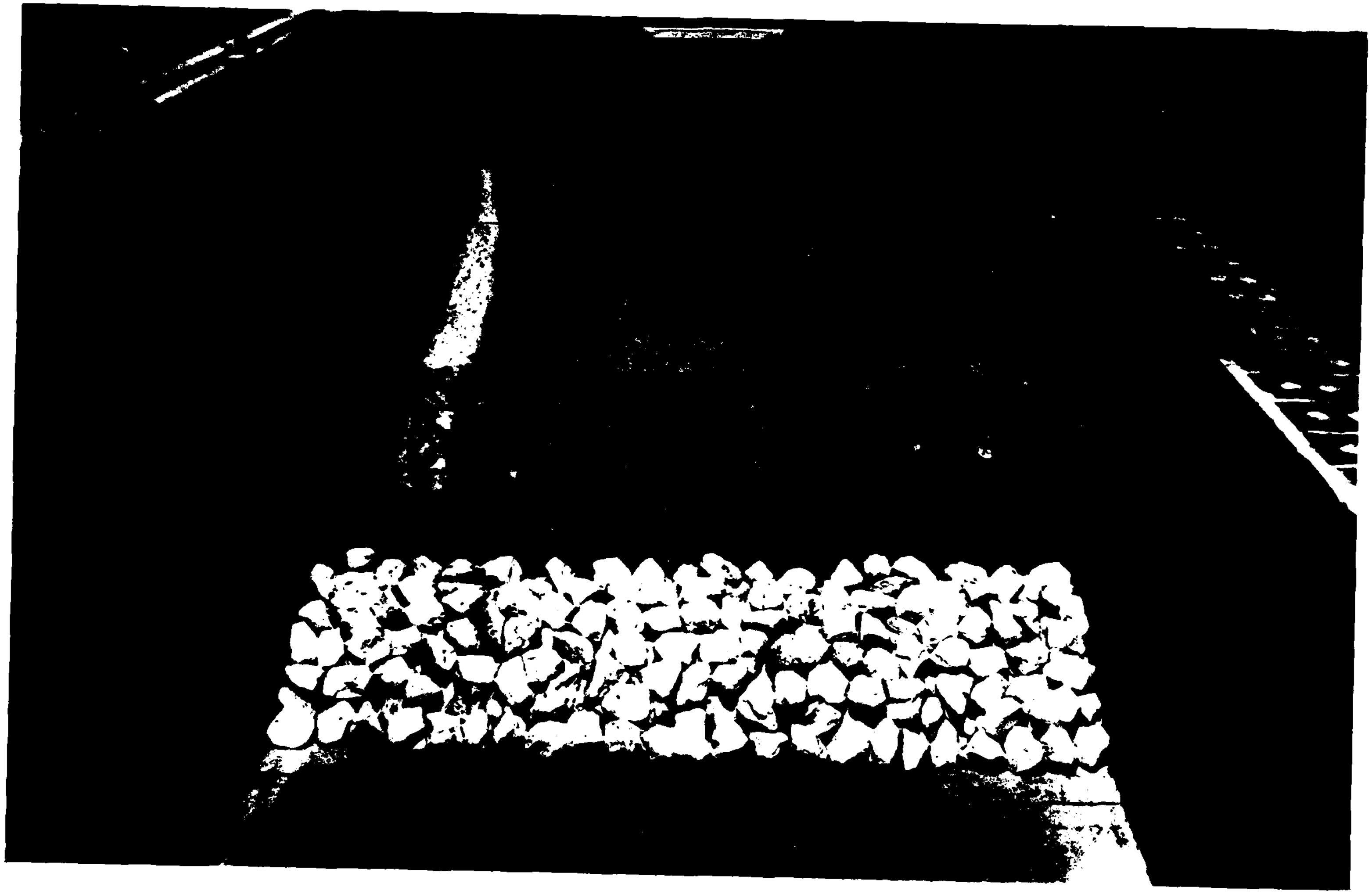


Photo 35. Sea-side view of Plan N-3 before testing, 1st test section

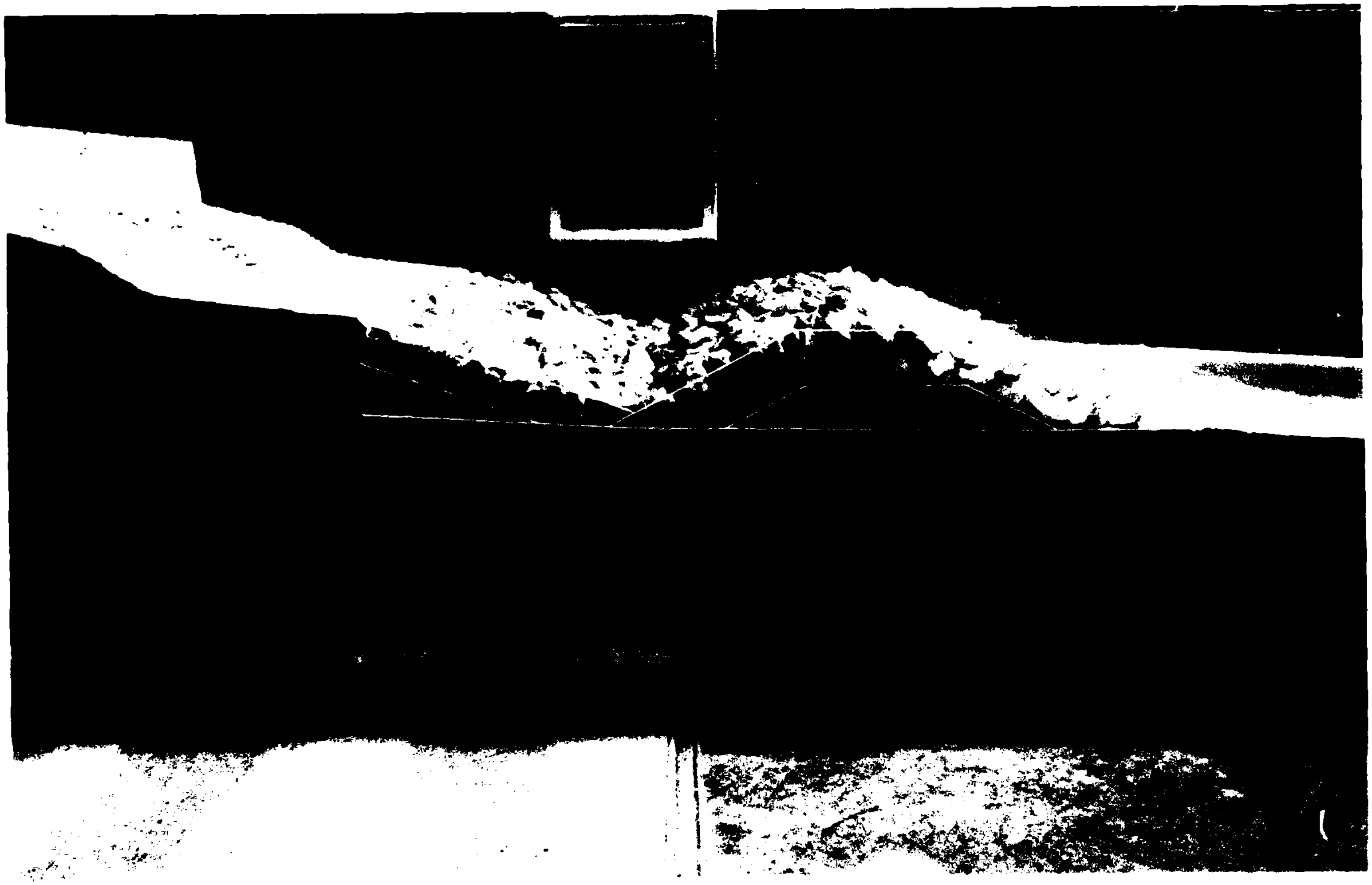


Photo 36. Side view of Plan N-3 after testing Hydrograph A (Plate 1 and Table 1), 1st test section

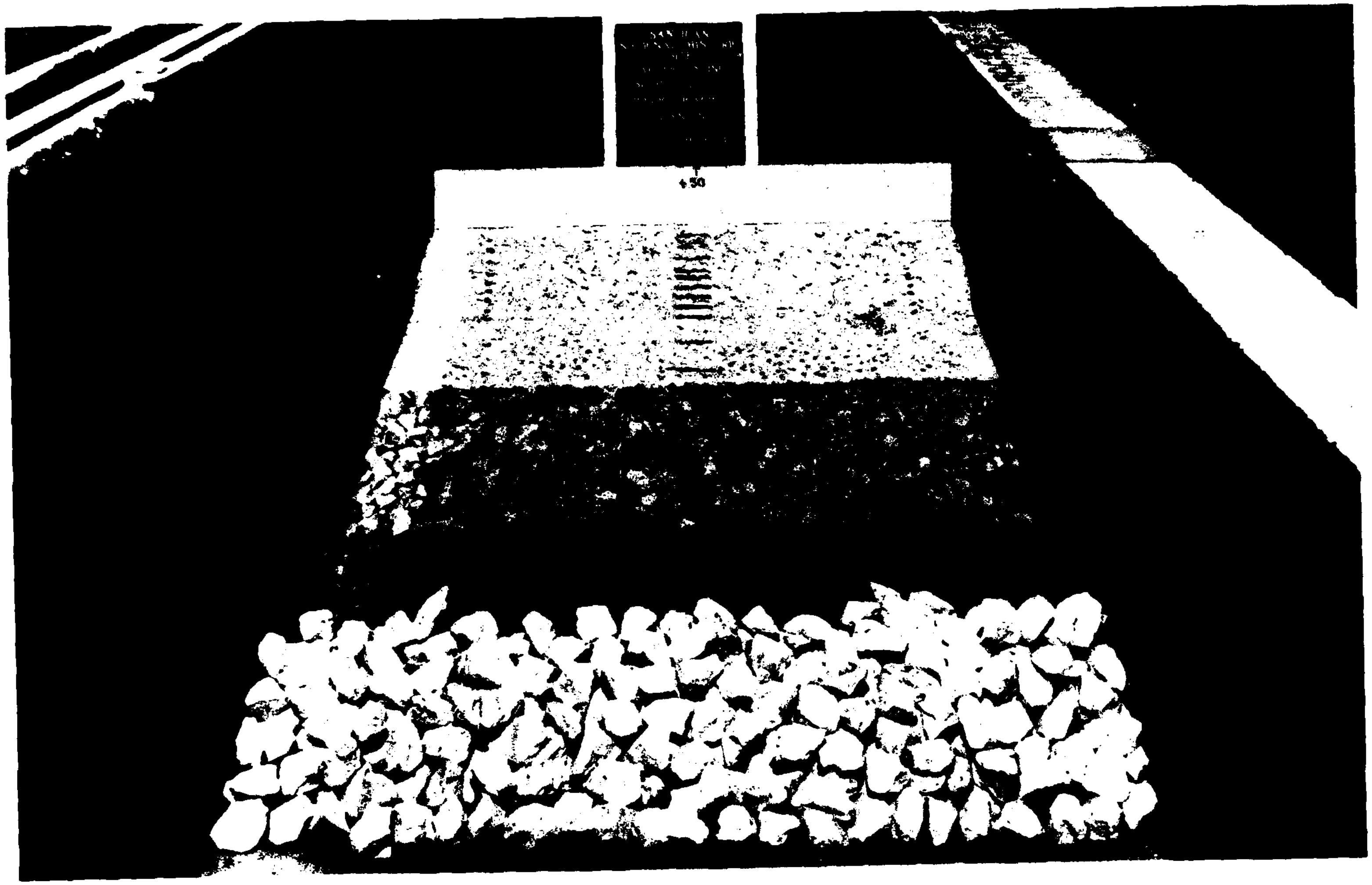


Photo 37. Sea-side view of Plan N-3 after testing Hyrograph A (Plate 1 and Table 1), 1st test section

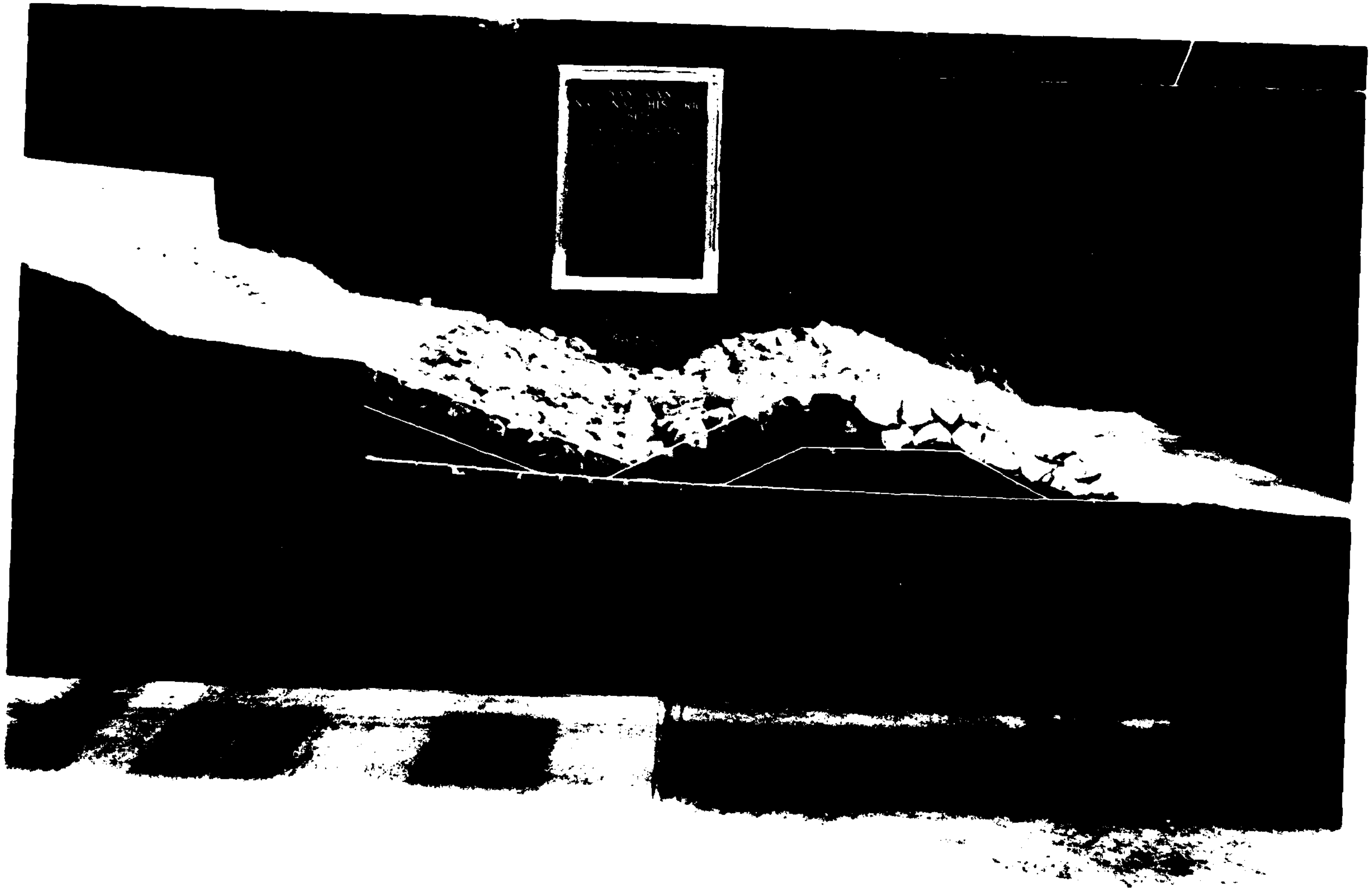


Photo 38. Side view of Plan N-3 after testing Hydrograph A (Plate 1 and Table 1), 2nd test section

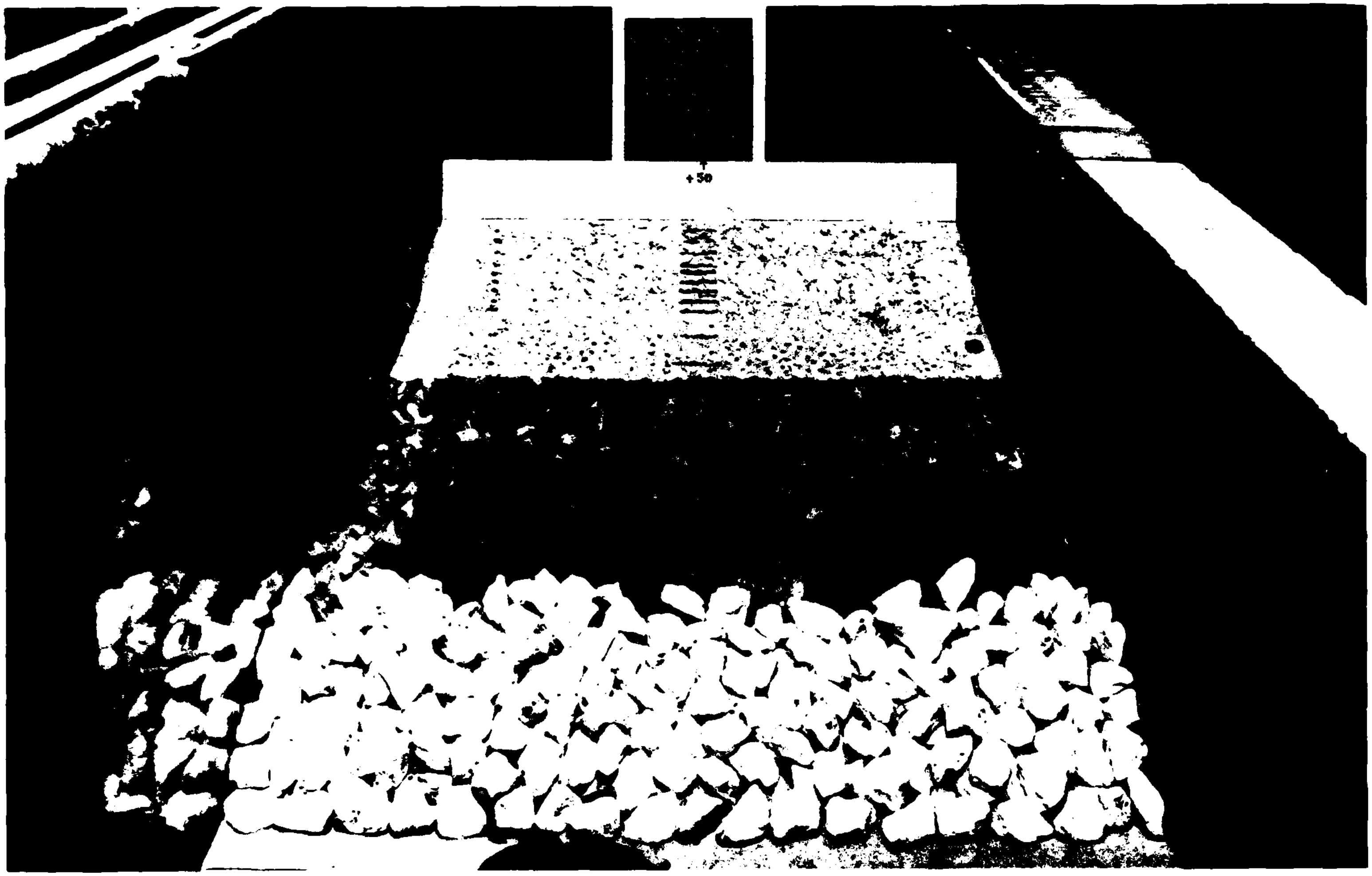


Photo 39. Sea-side view of Plan N-3 after testing Hydrograph A (Plate 1 and Table 1), 2nd test section

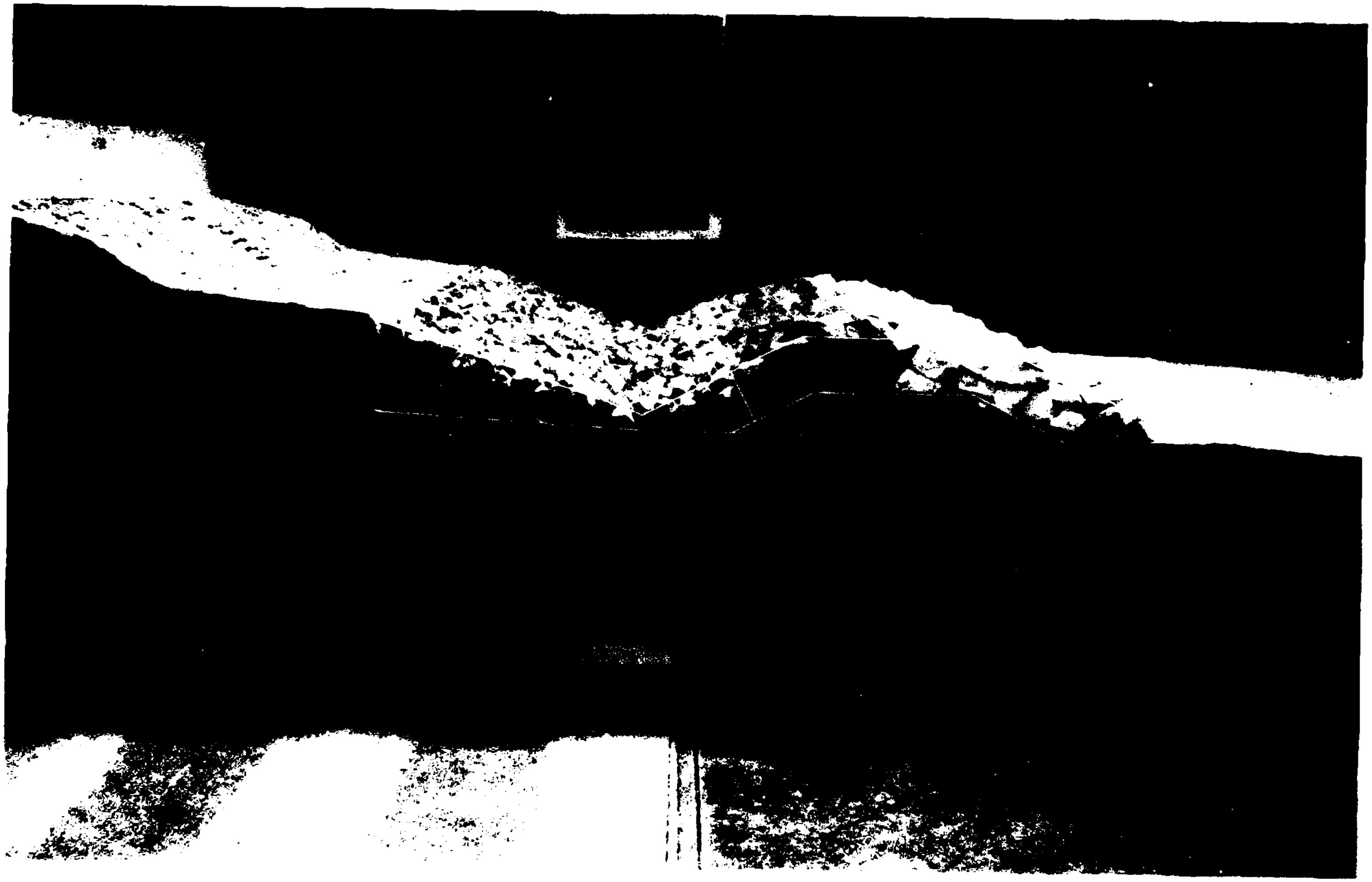


Photo 40. Side view of Plan N-3-A before testing, 1st test section

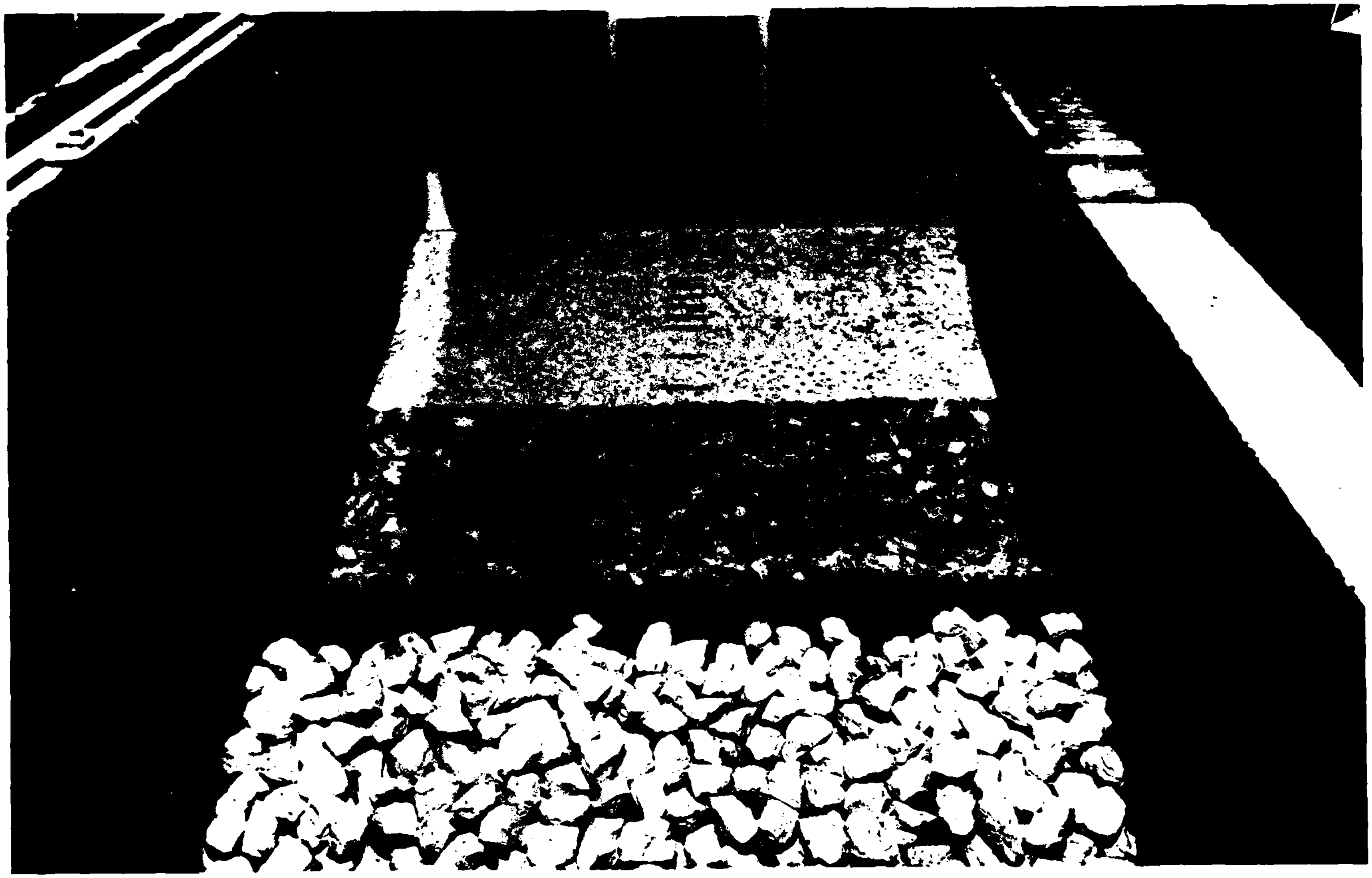


Photo 41. Sea-side view of Plan N-3-A before testing, 1st test section

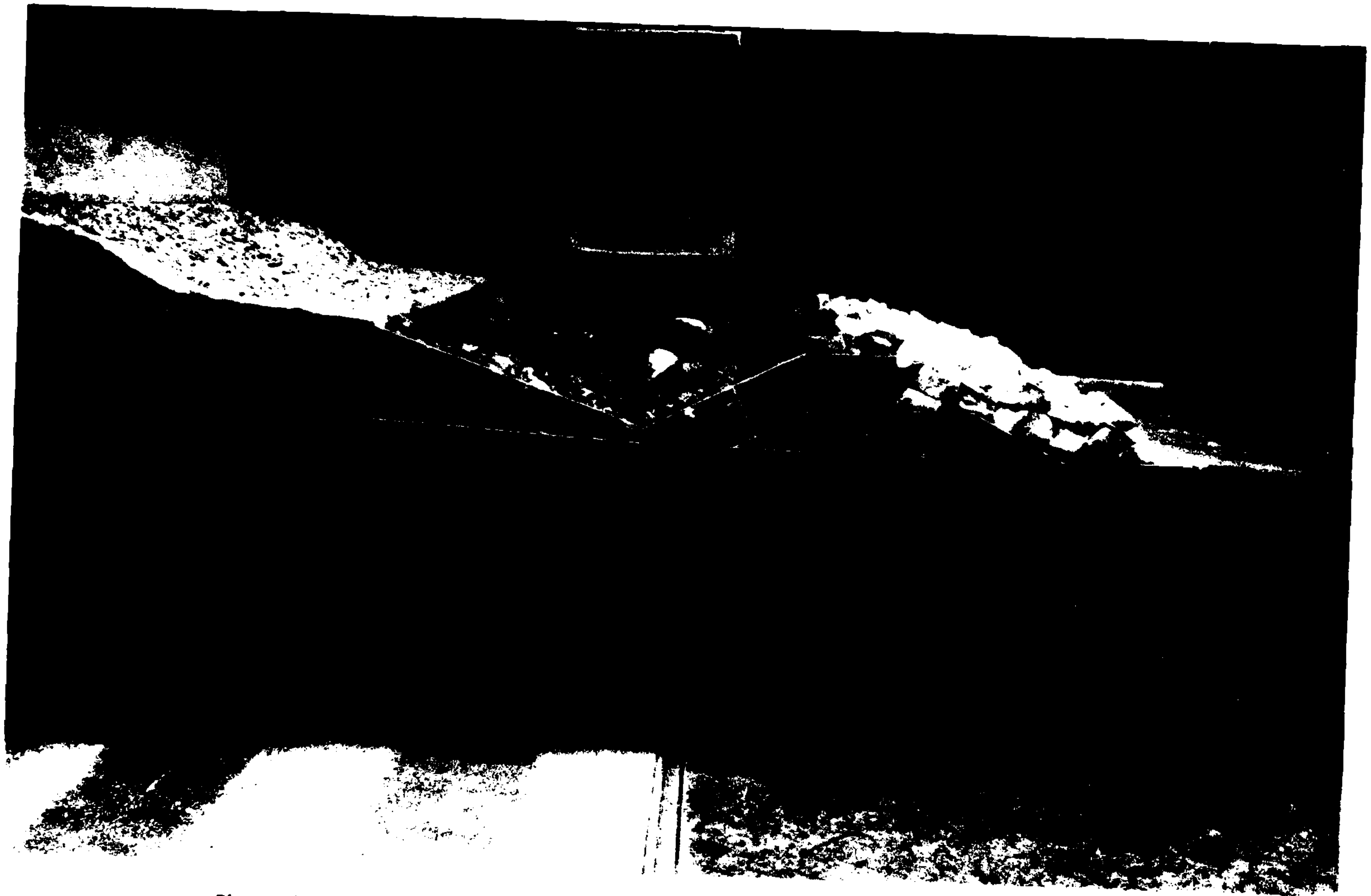


Photo 42. Side view of Plan N-3-A after testing Hydrograph A (Plate 1 and Table 1), 1st test section

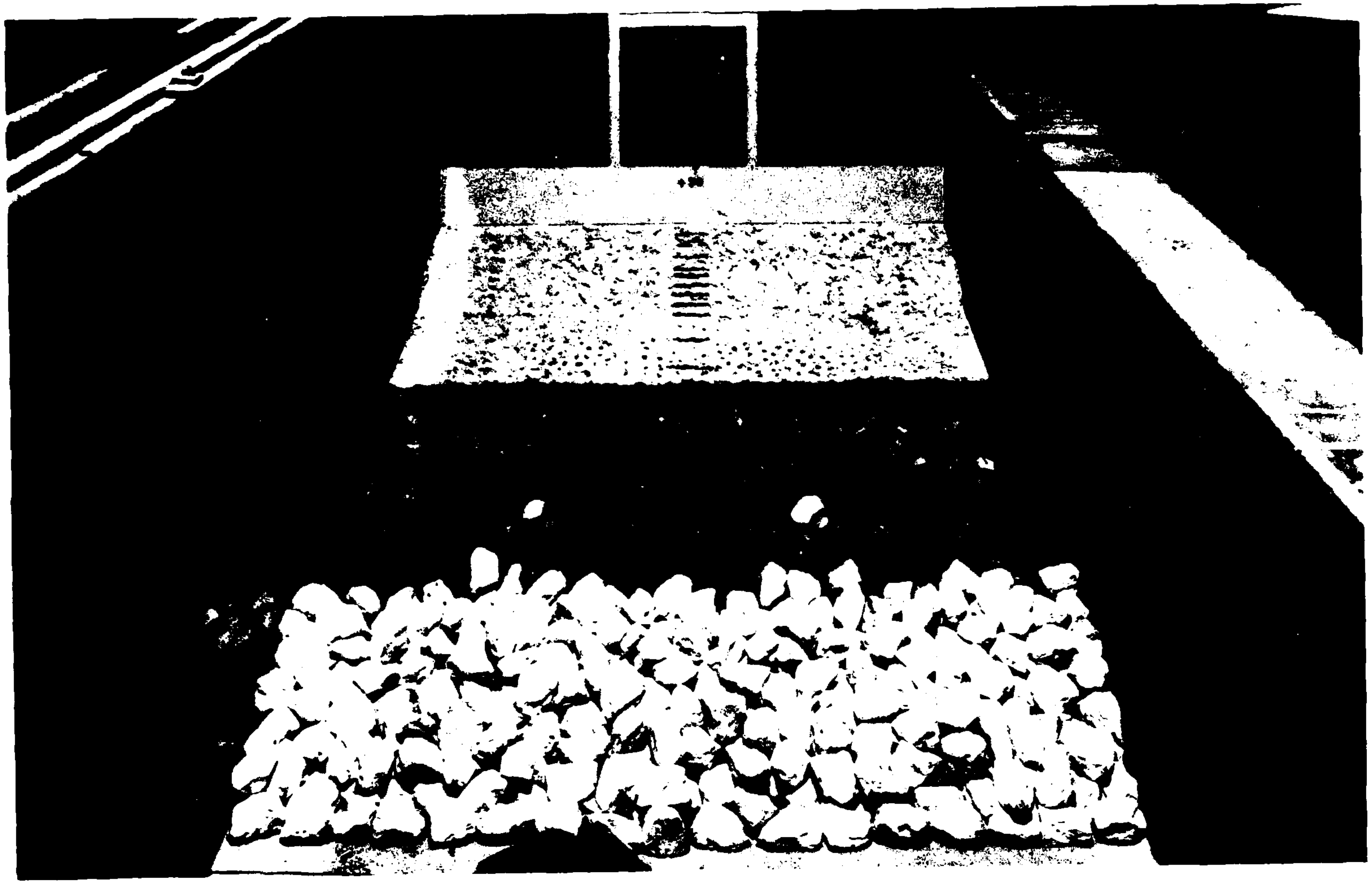


Photo 43. Sea-side view of Plan N-3-A after testing Hydrograph A (Plate 1 and Table 1), 1st test section

Photo 44. Side view of Plan N-3-A after testing Hydrograph A (Plate 1 and Table 1), 2nd test section

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SEP 81 D G MARKLE
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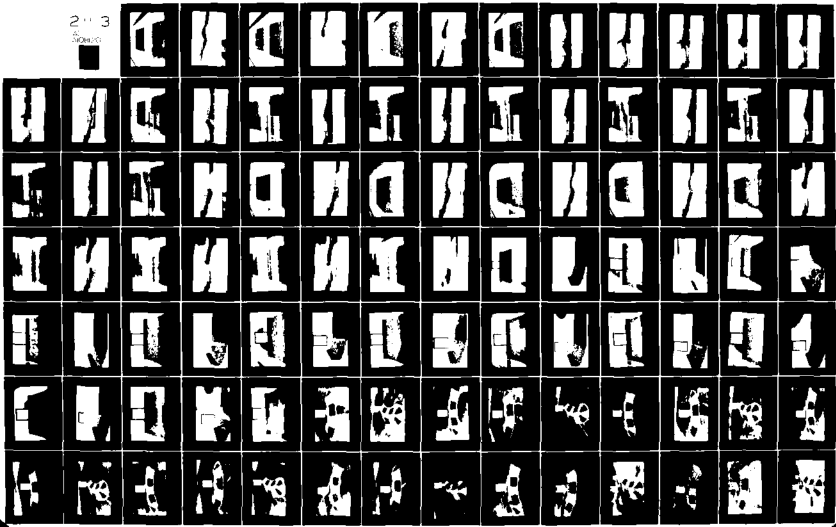

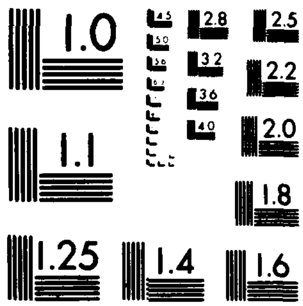

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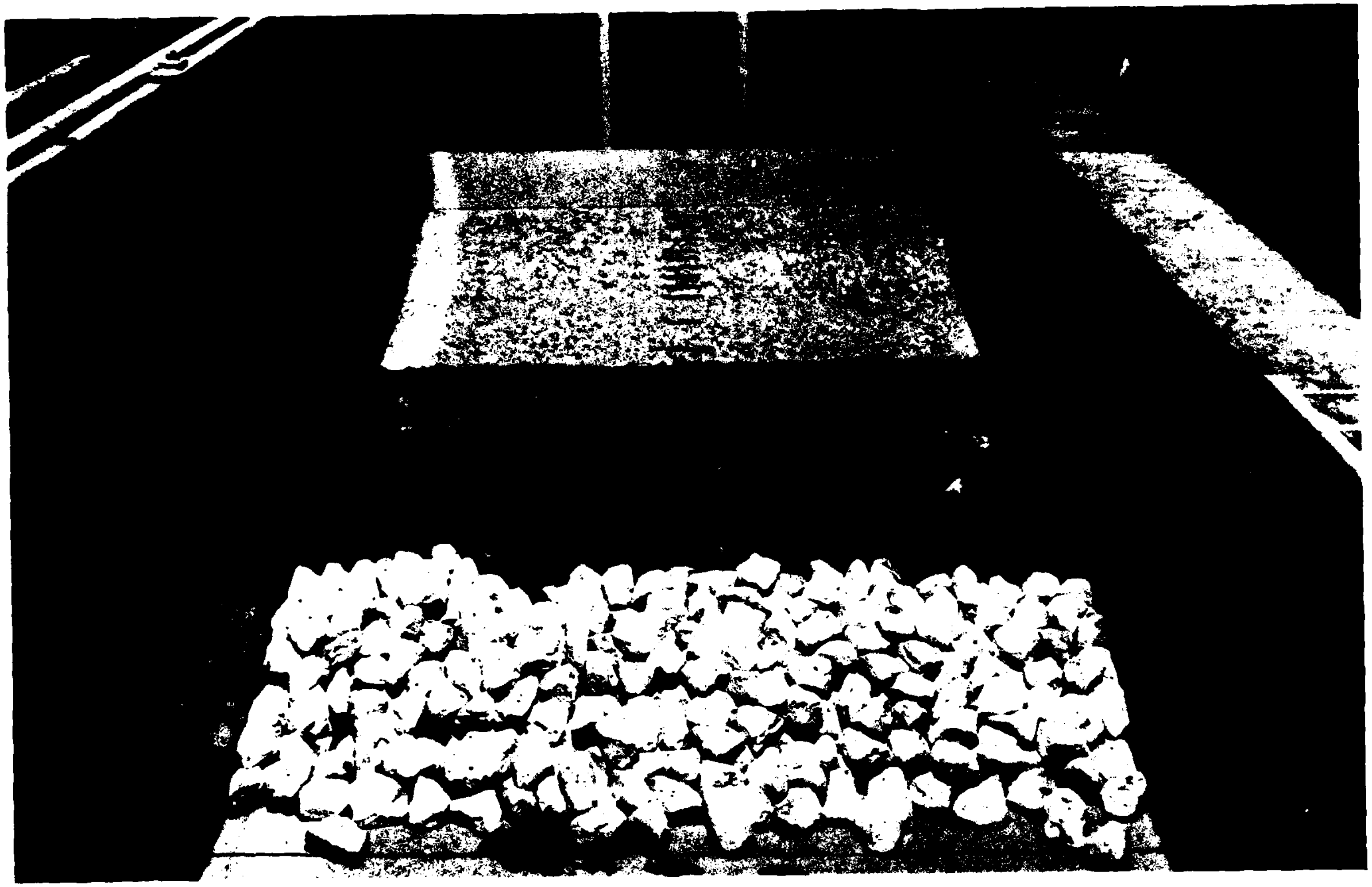

Photo 45. Sea-side view of Plan N-3-A after testing Hydrograph A (Plate 1 and Table 1), 2nd test setion

Photo 46. Side view of Plan N-4 before testing, 1st test section

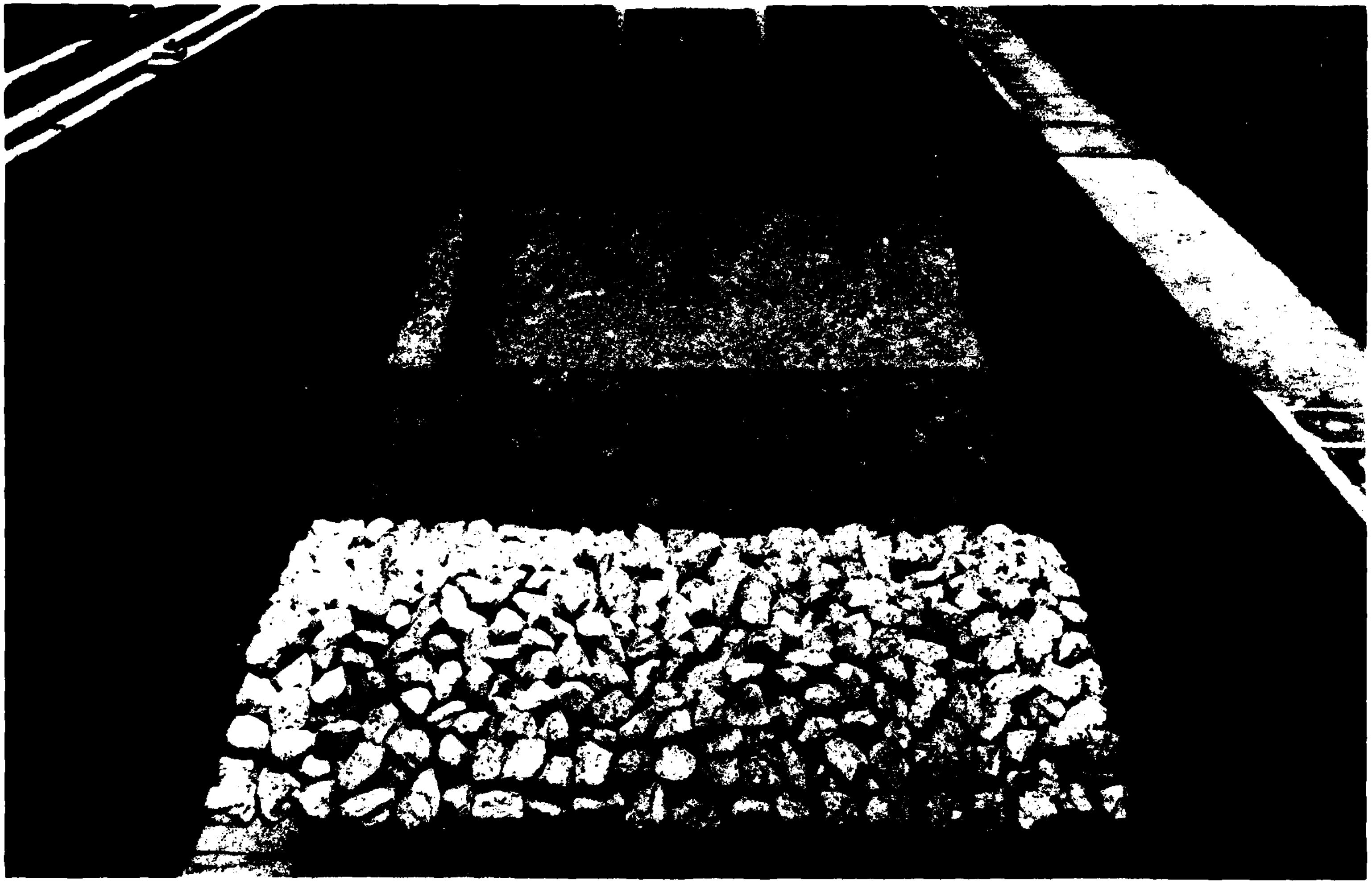

Photo 47. Sea-side view of Plan N-4 before testing, 1st test section

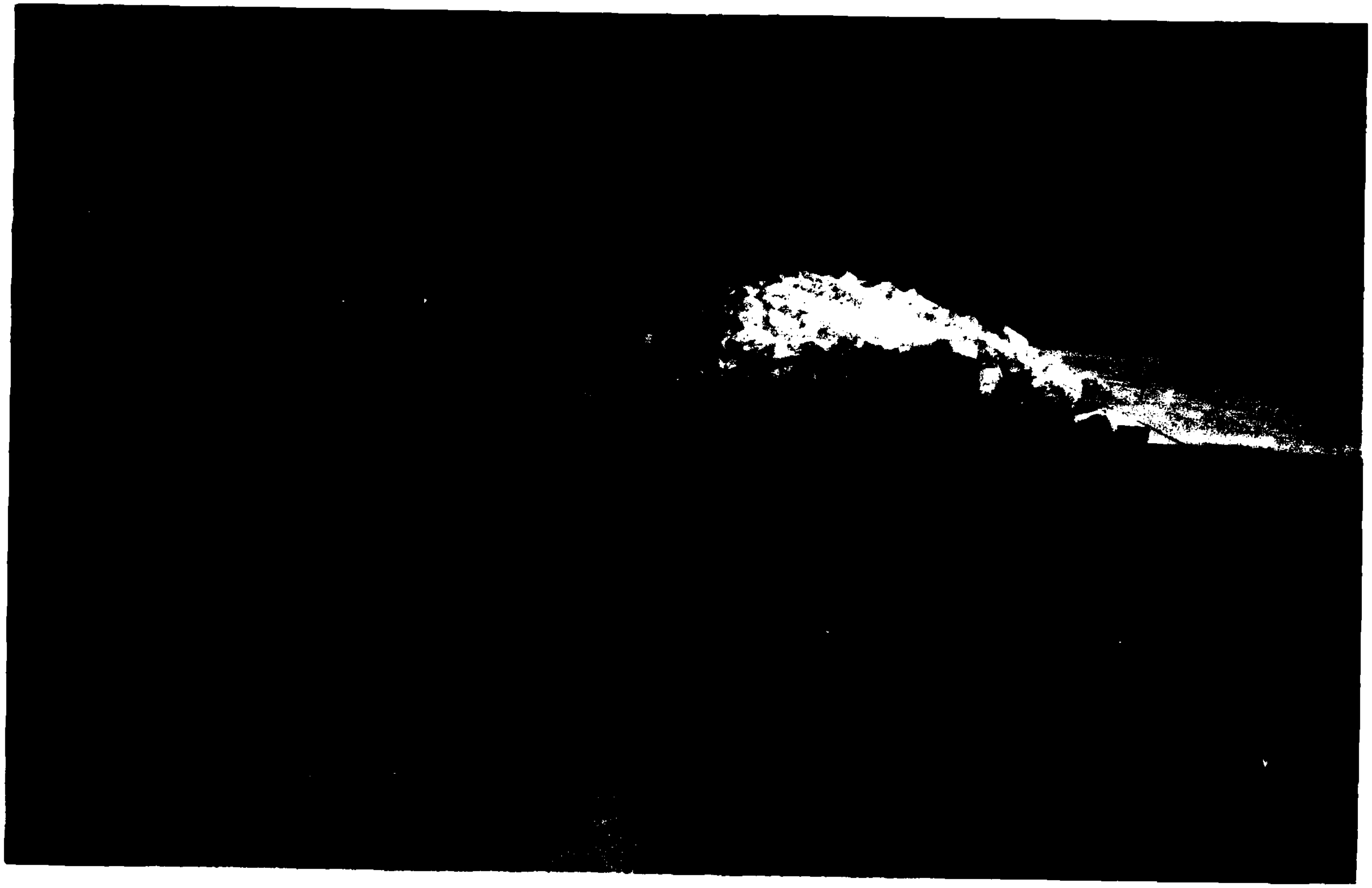

Photo 48. Side view of Plan N-4 after testing Hydrograph A (Plate 1 and Table 1), 1st test section

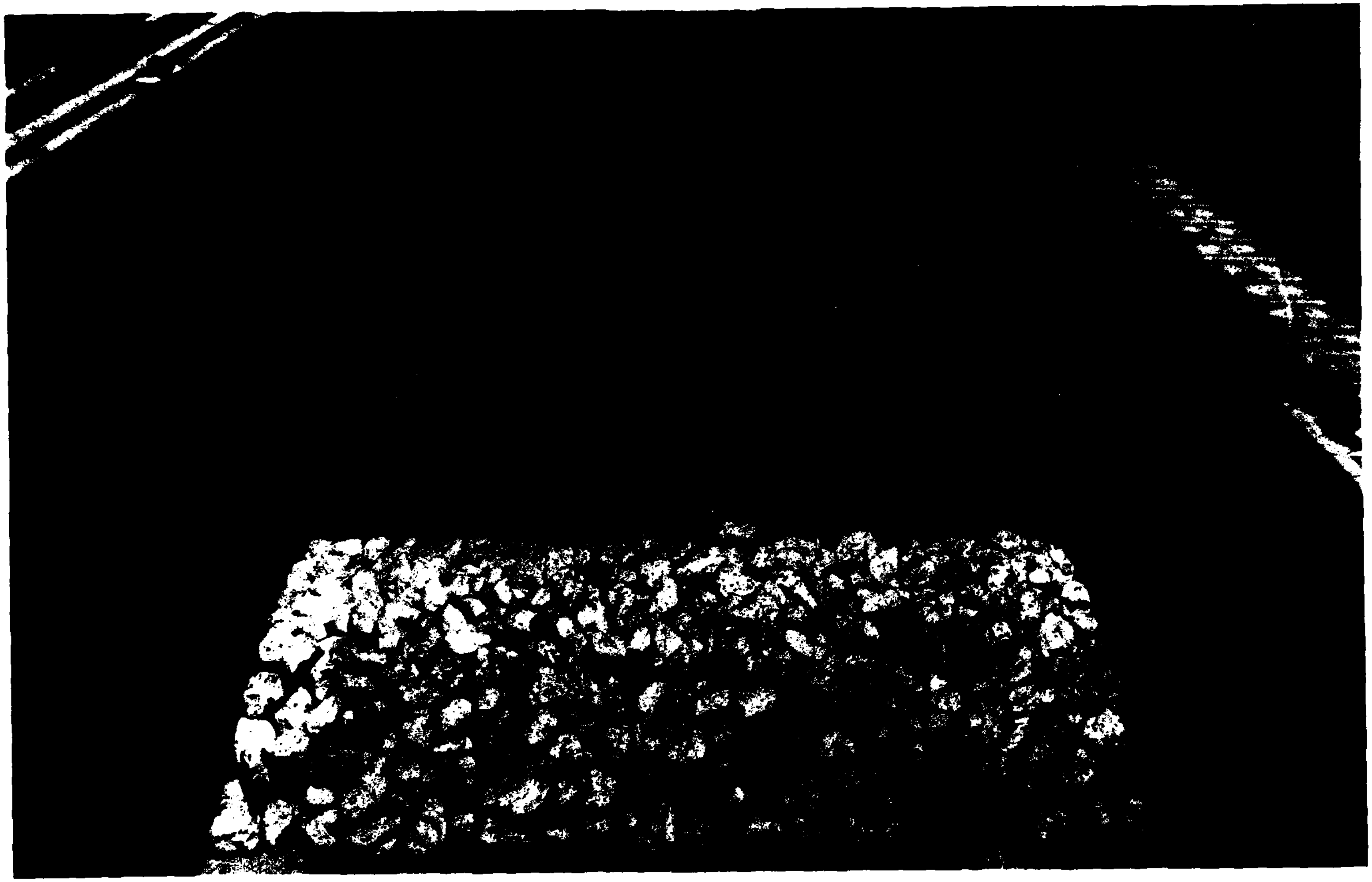

Photo 49. Sea-side view of Plan N-4 after testing Hydrograph A (Plate 1 and Table 1), 1st test section

Photo 50. Side view of Plan N-4 after testing Hydrograph A (Plate 1 and Table 1), 2nd test section

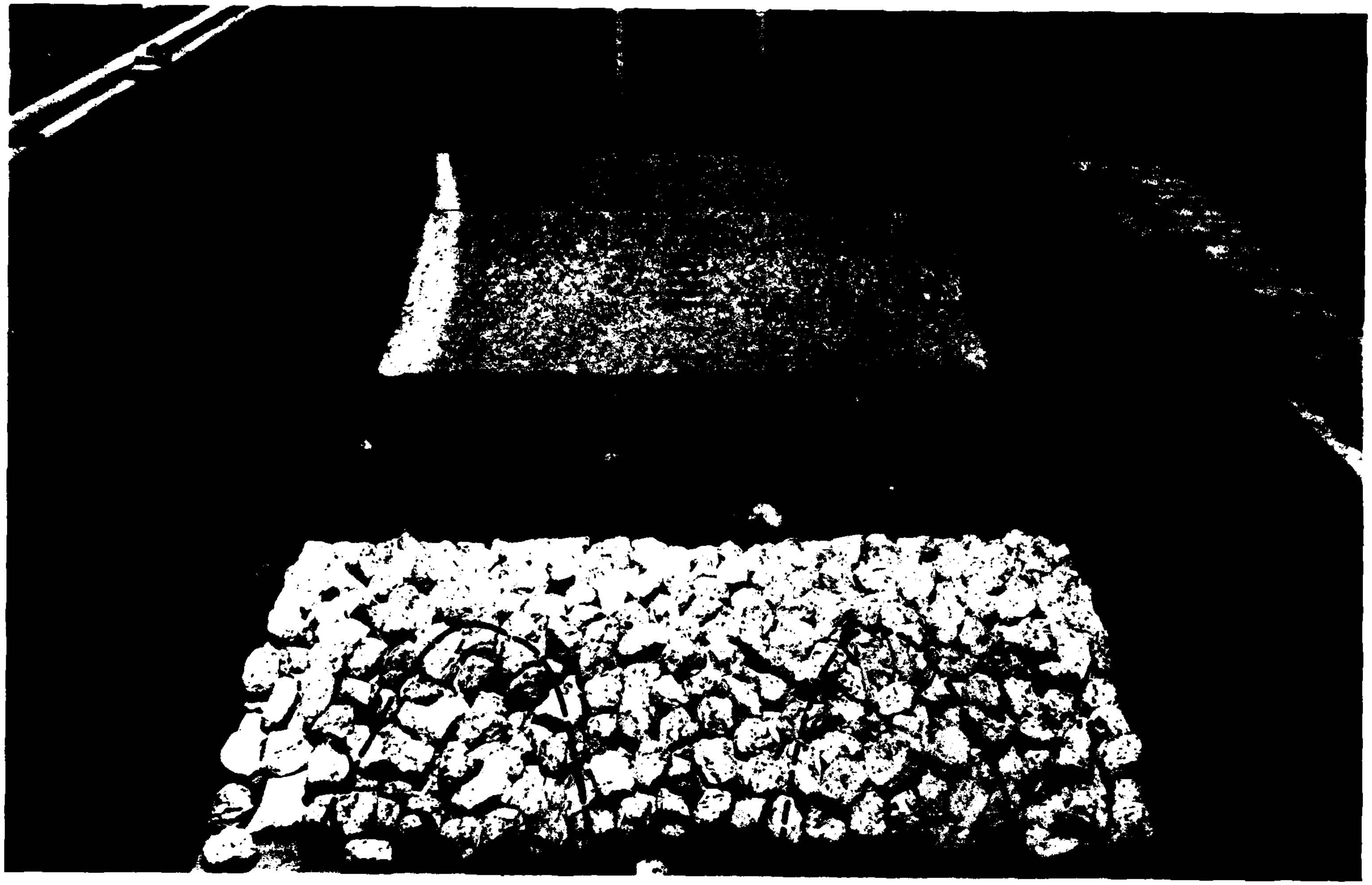

Photo 51. Sea-side view of Plan N-4 after testing Hydrograph A (Plate 1 and Table 1), 2nd test section

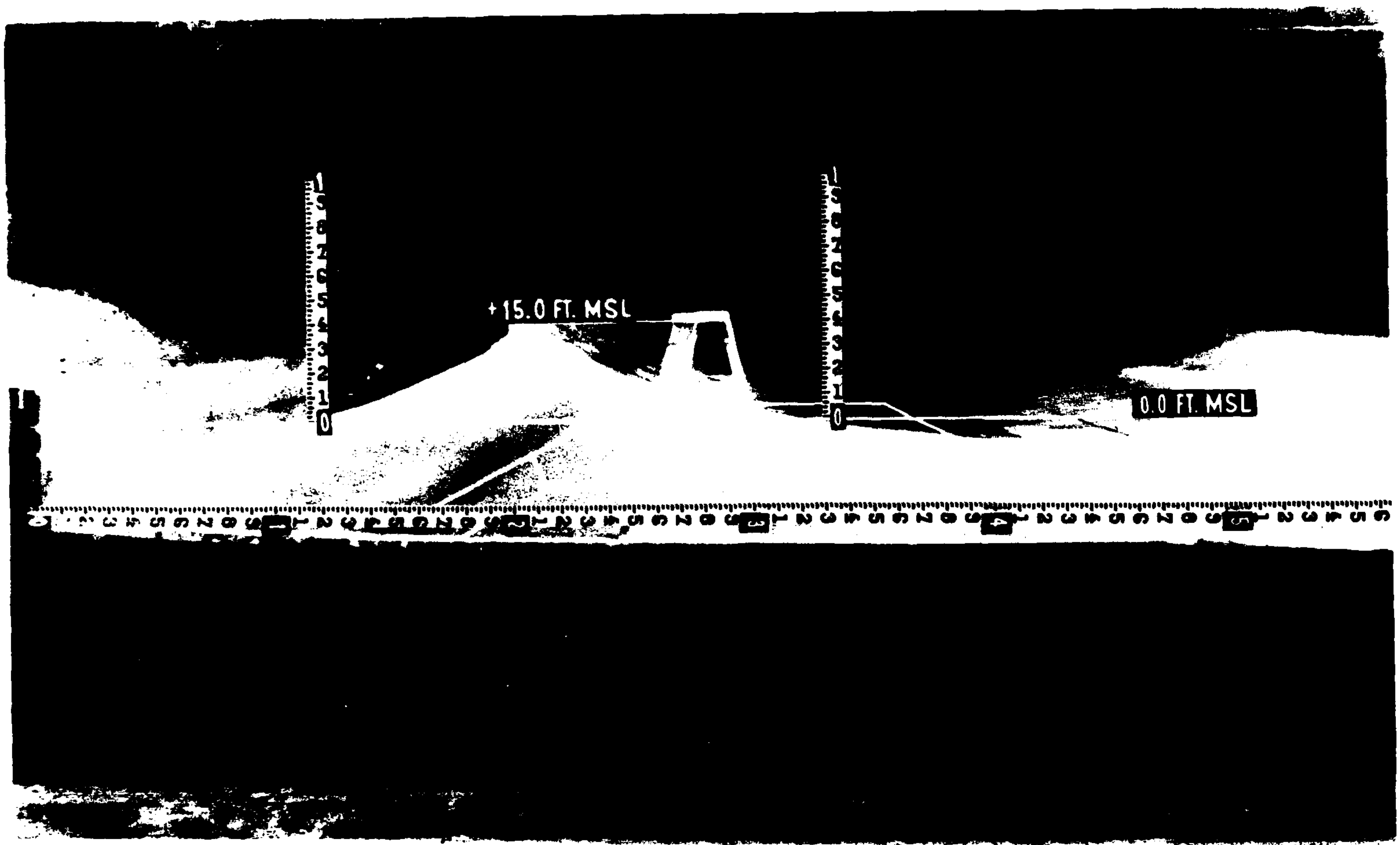

Photo 52. Side view of trestle on north slope, 7.0-sec, 10.0-ft wave at +1.9 swl

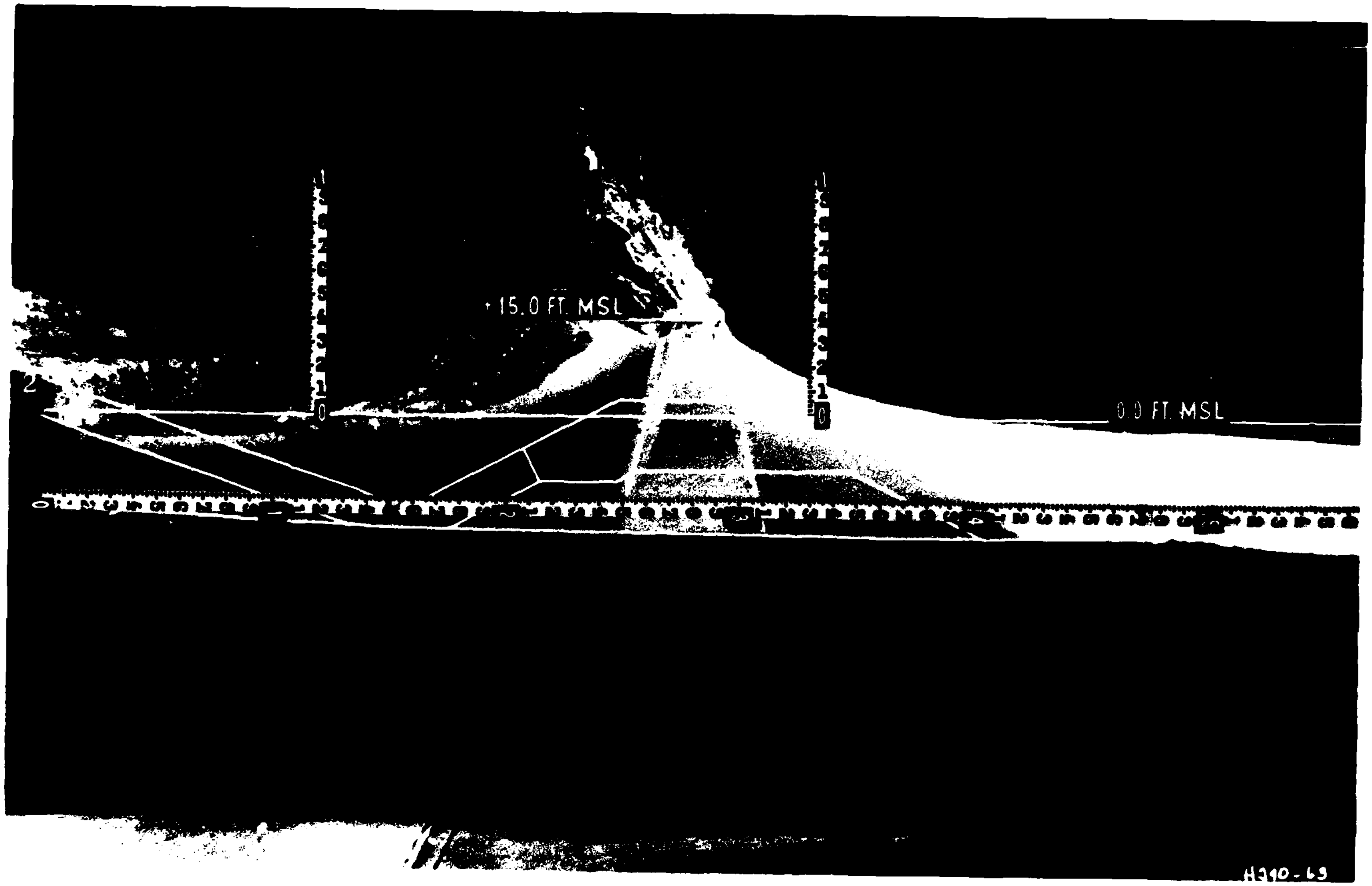

Photo 53. Side view of trestle on north slope, 8.0-sec, 12.0-ft wave at +1.9 swl

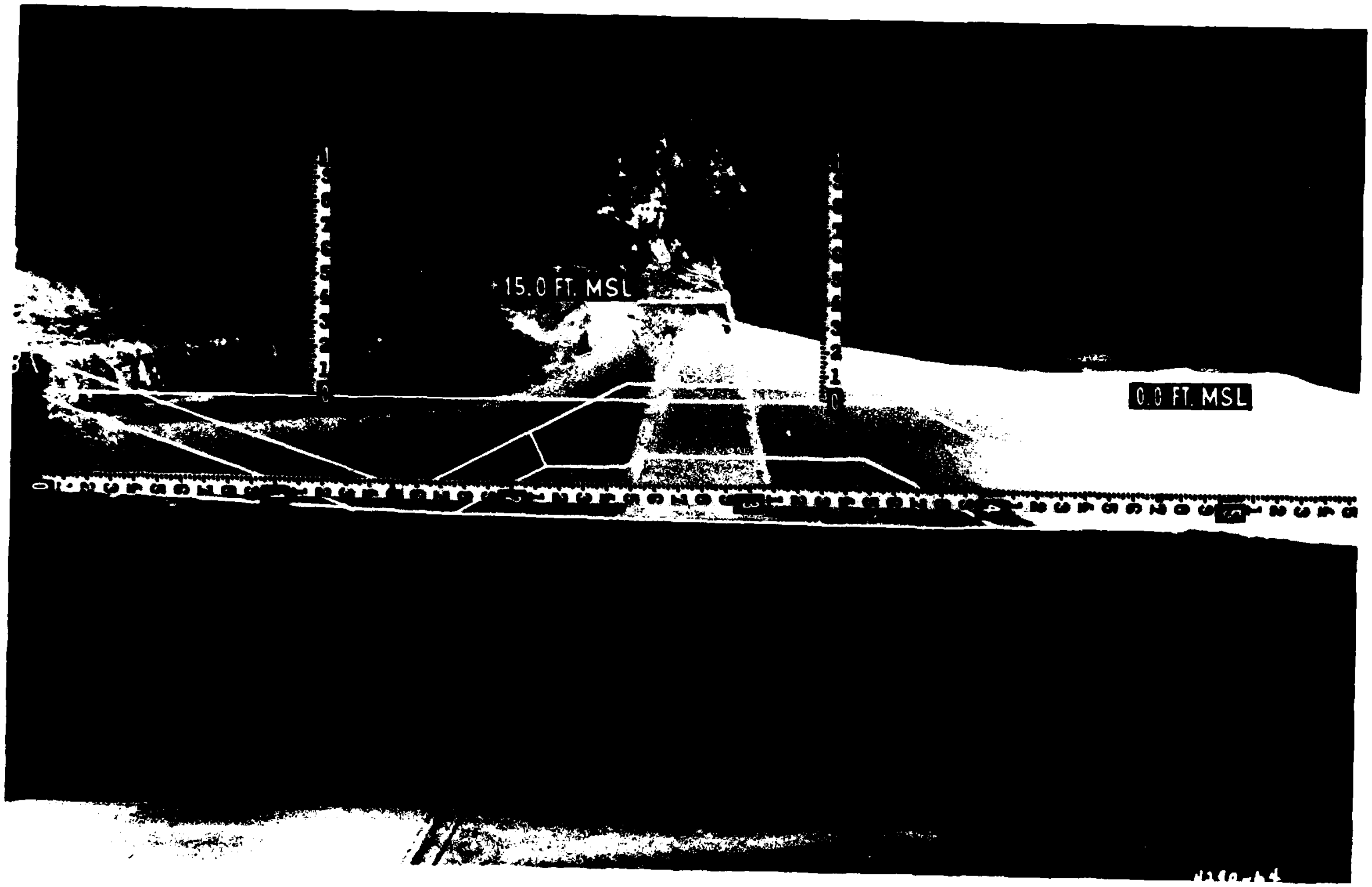

Photo 54. Side view of trestle on north slope, 17.0-sec, 20.0-ft wave at +1.9 swl

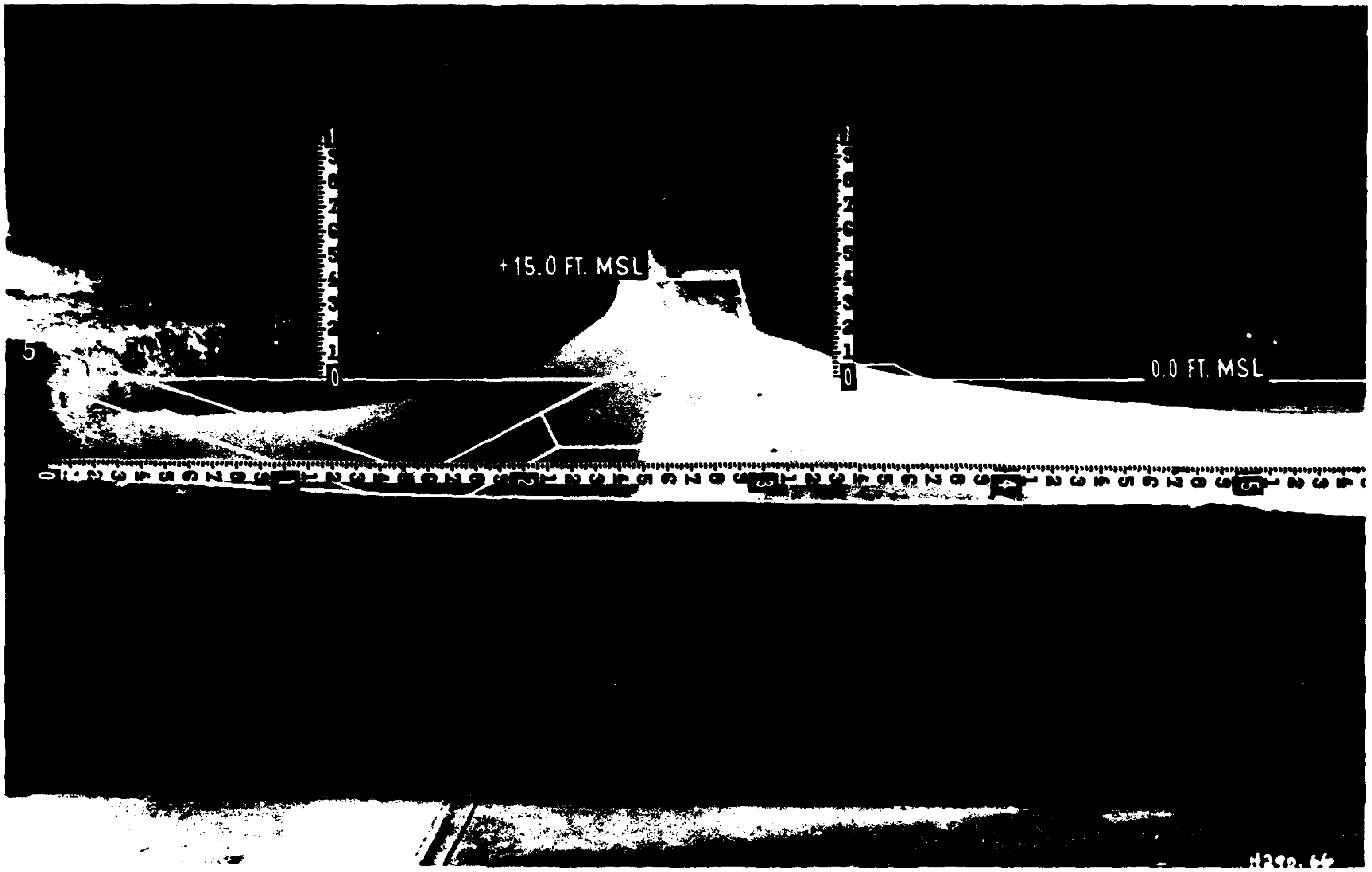

Photo 55. Side view of trestle on north slope, 8.5-sec, 12.0-ft wave at 0.0 swl

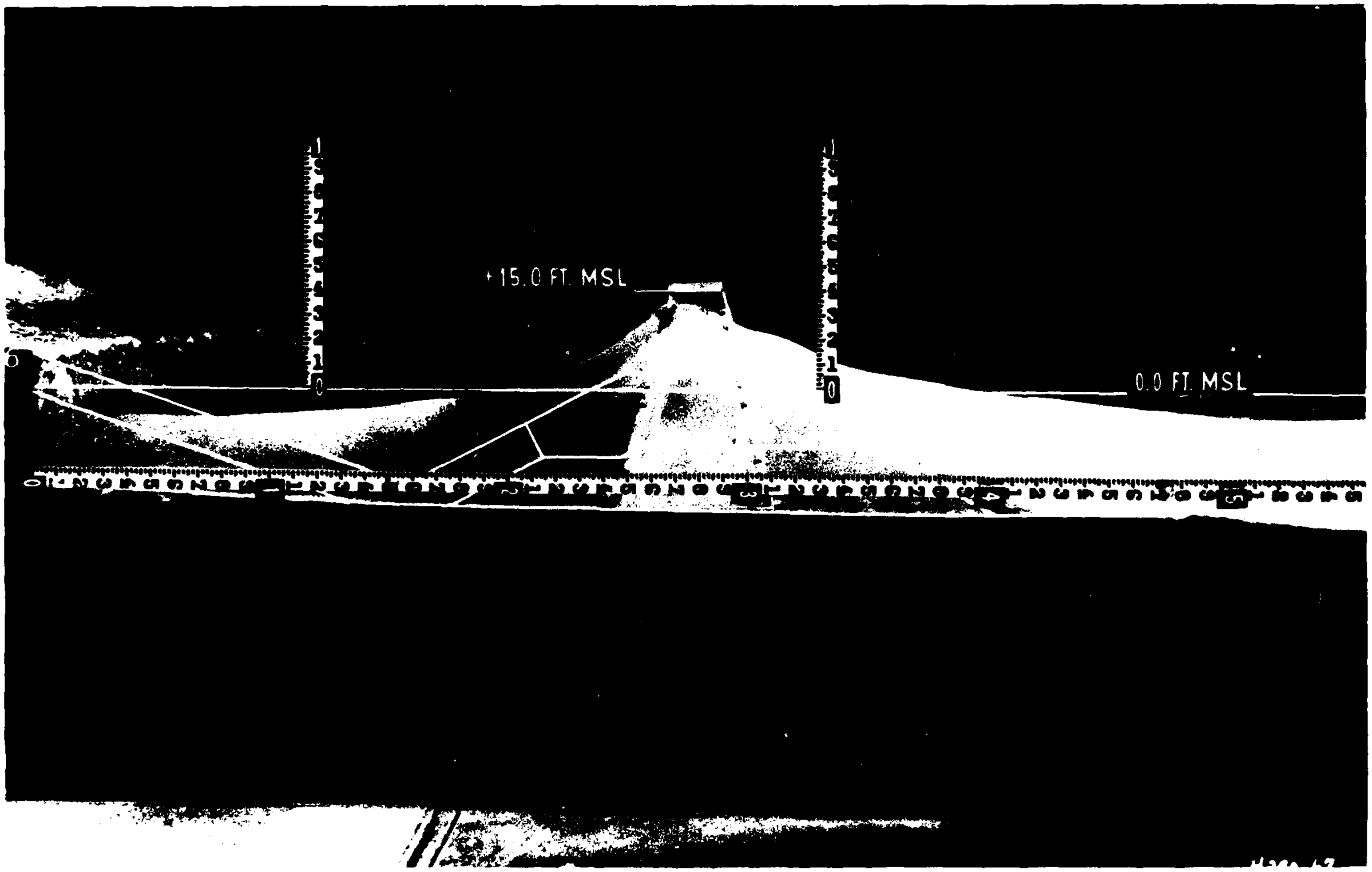

Photo 56. Side view of trestle on north slope, 9.0-sec, 10.0-ft wave at 0.0 swl

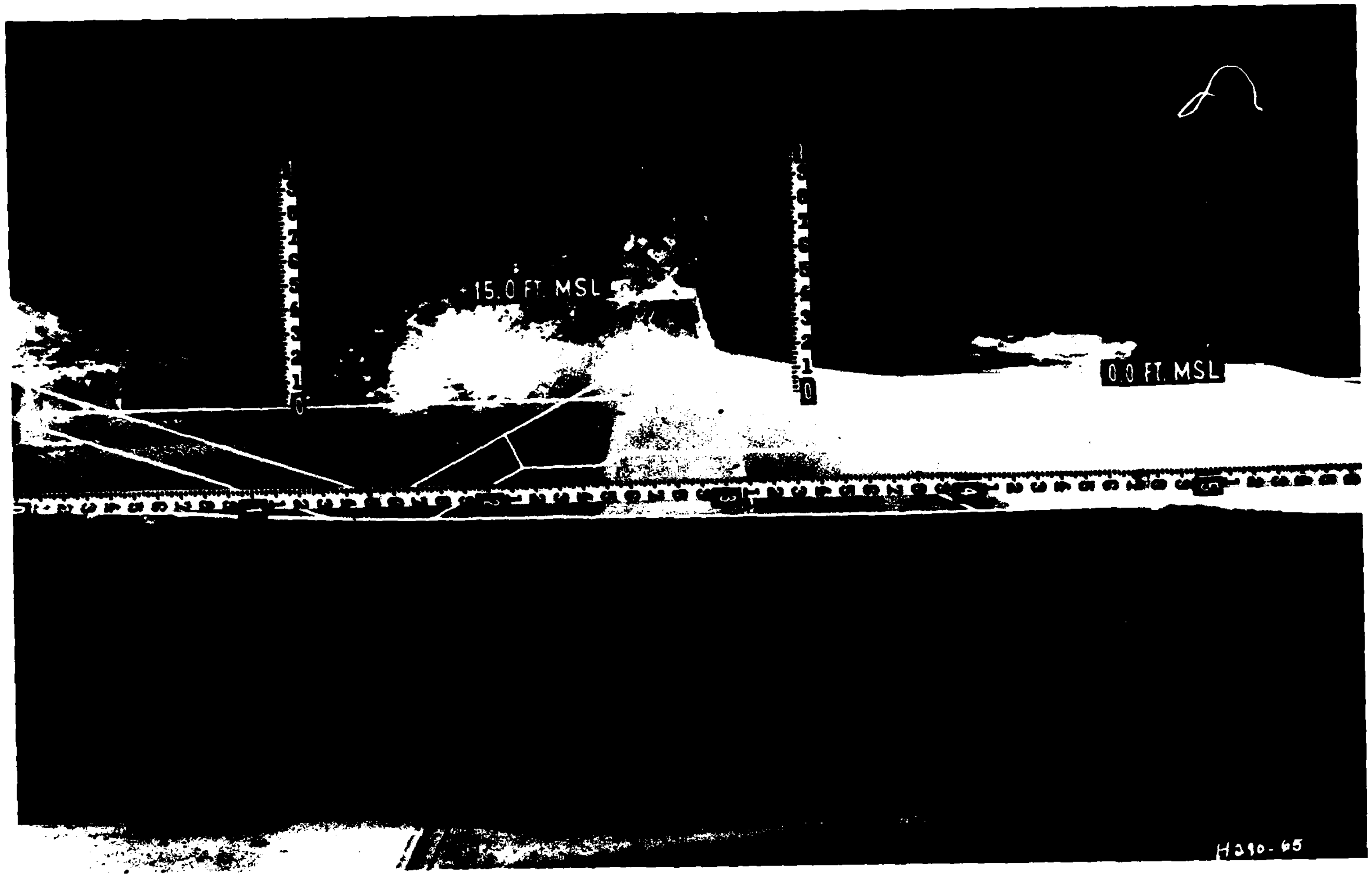

Photo 57. Side view of trestle on north slope, 17.0-sec, 20.0-ft wave at 0.0 swl

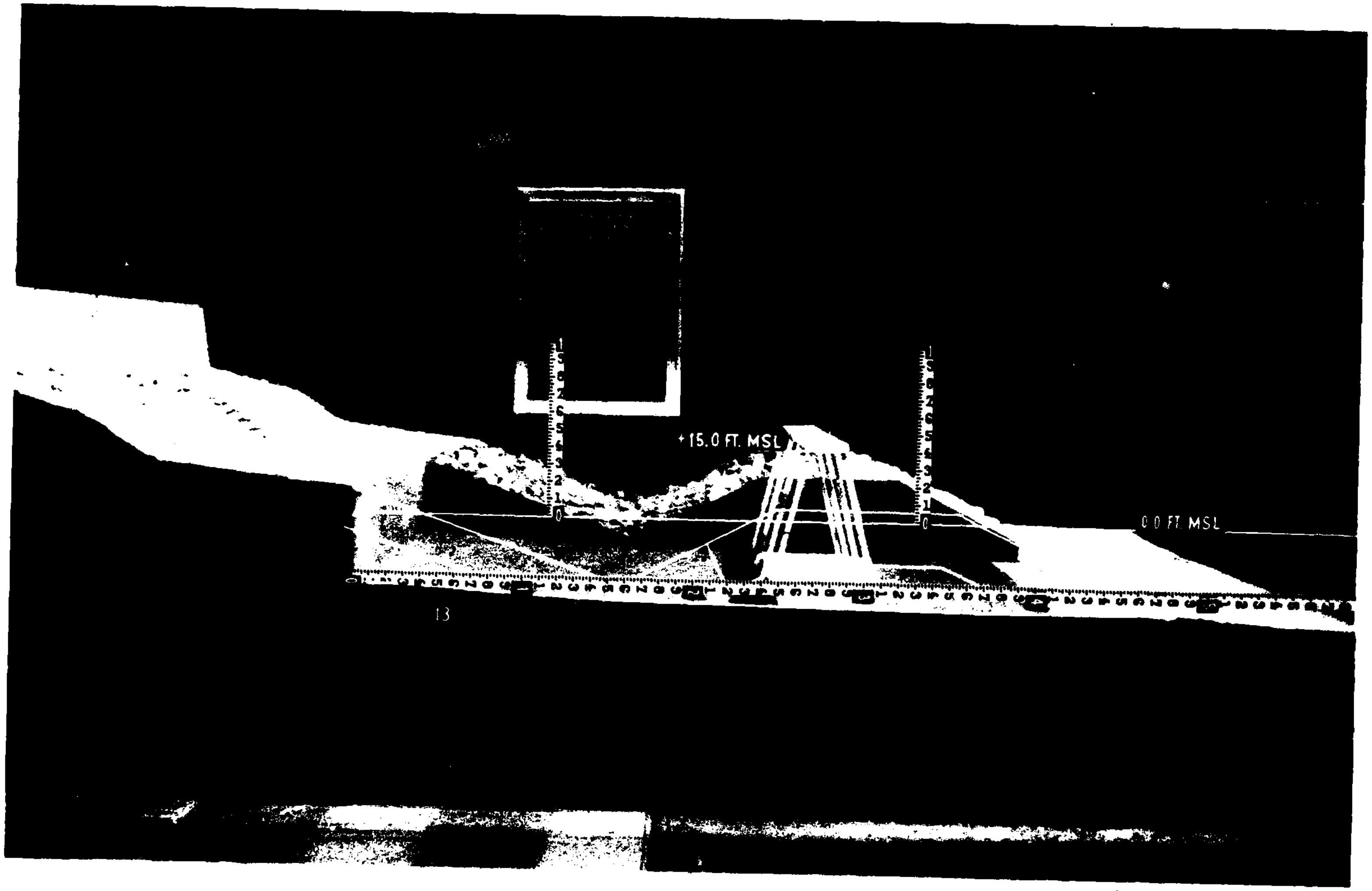

Photo 58. Side view of Plan N-3-A with trestle

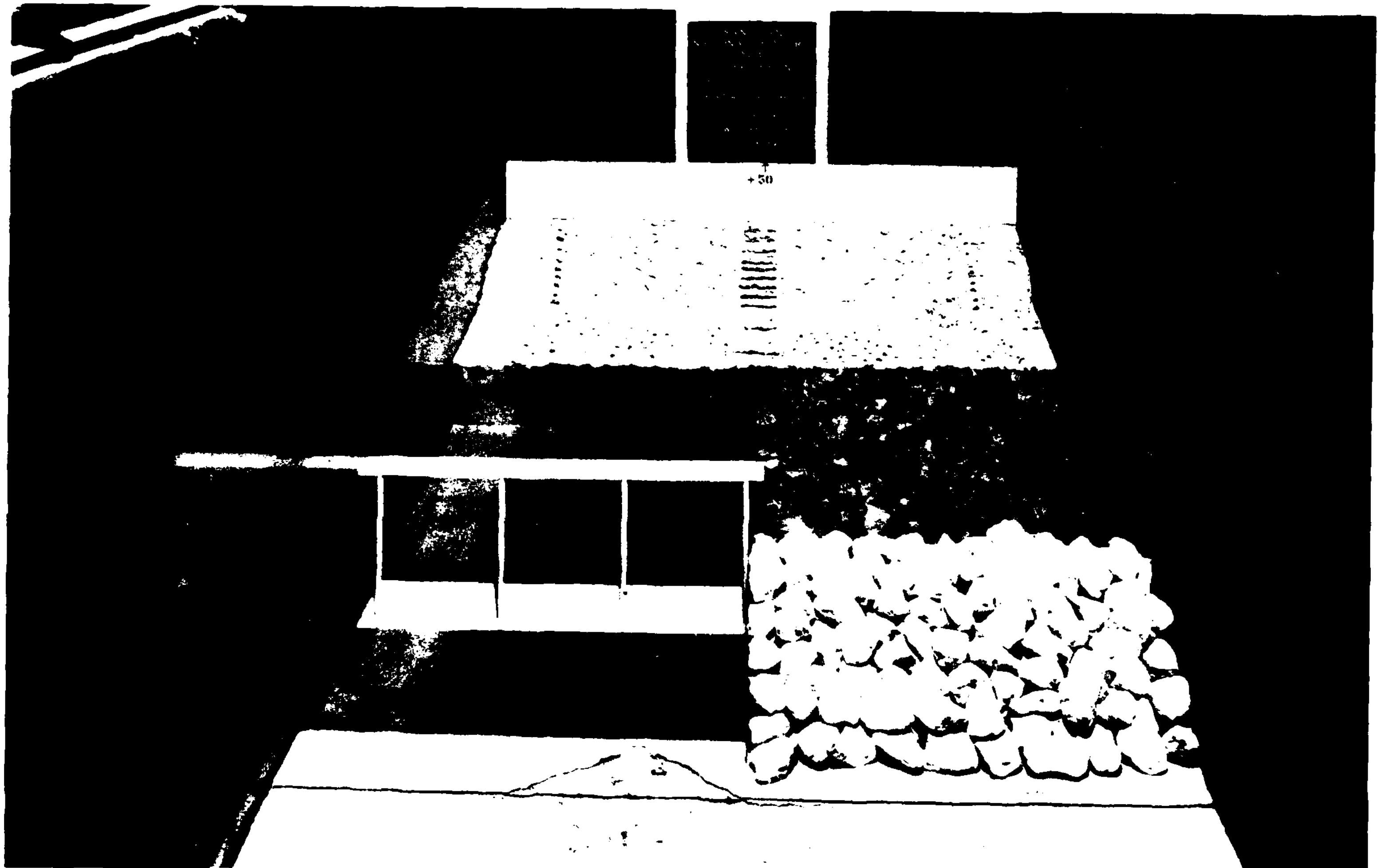

Photo 59. Sea-side view of Plan N-3-A with trestle

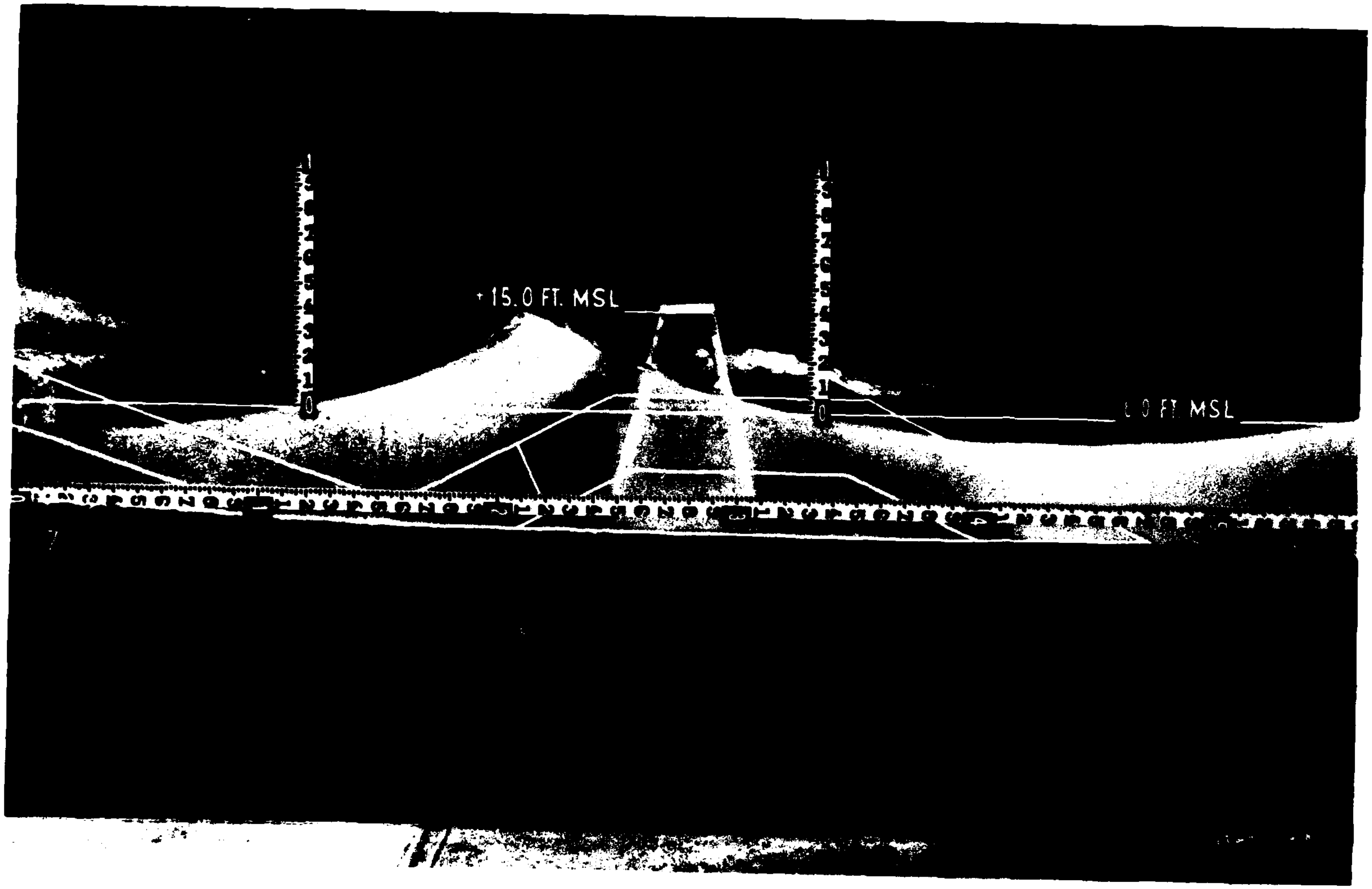

Photo 60. Side view of Plan N-3-A with trestle on north slope, 7.0-sec, 10.0-ft wave at +1.9 swl

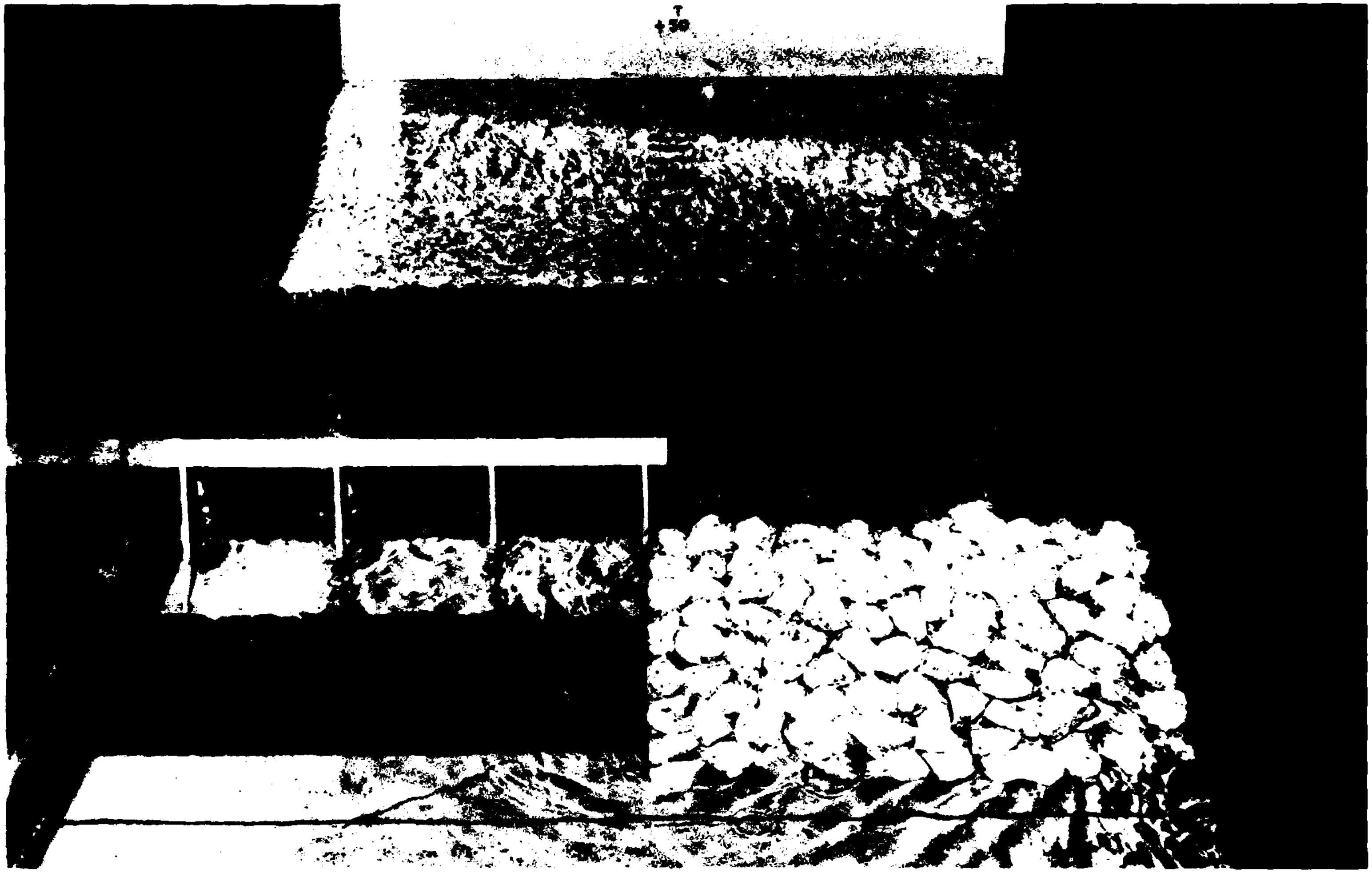

Photo 61. Sea-side view of Plan N-3-A with trestle on north slope, 7.0-sec, 10.0-ft wave at +1.9 swl

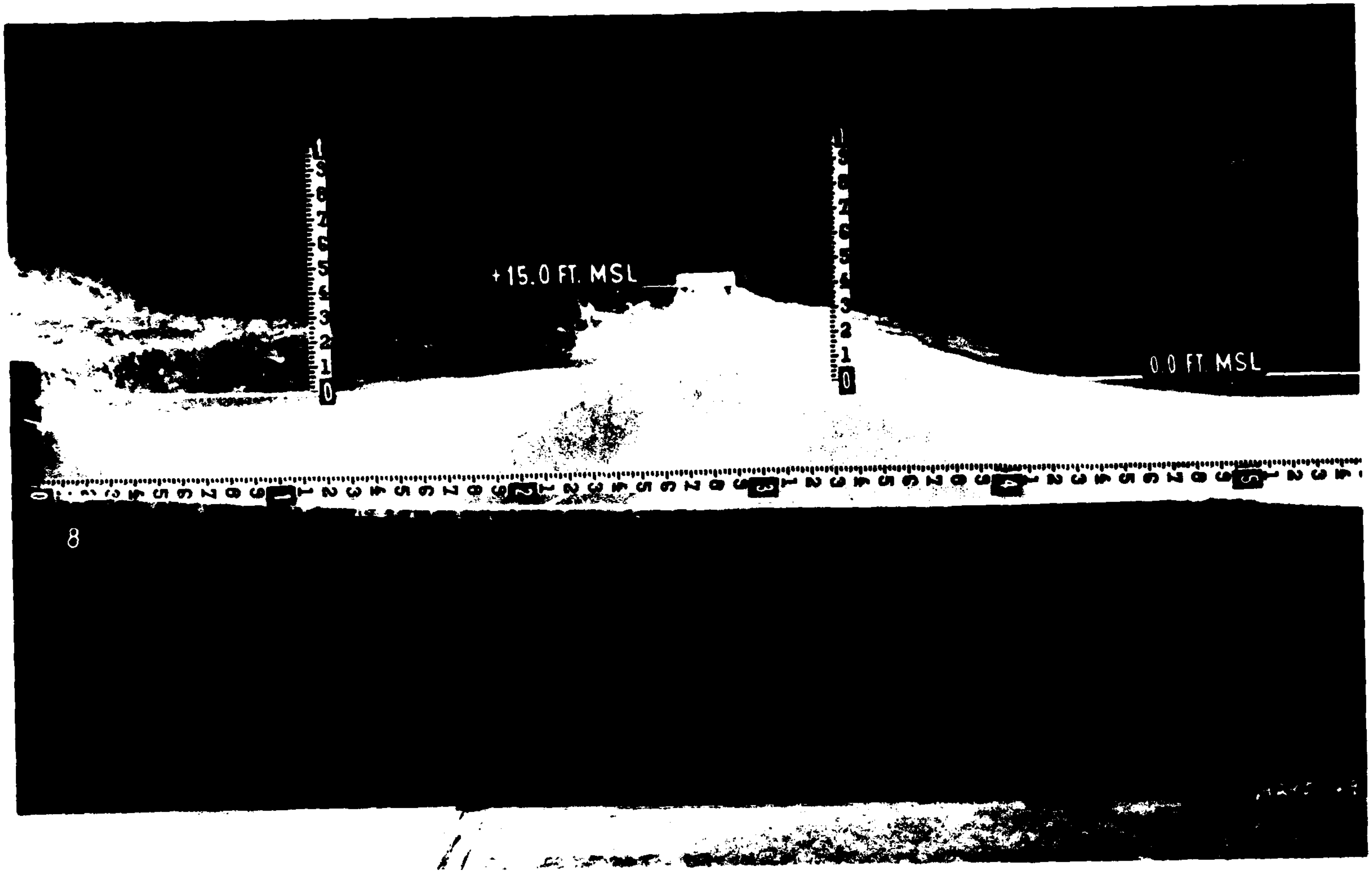

Photo 62. Side view of Plan N-3-A with trestle on north slope, 8.0-sec, 12.0-ft wave at +1.9 swl

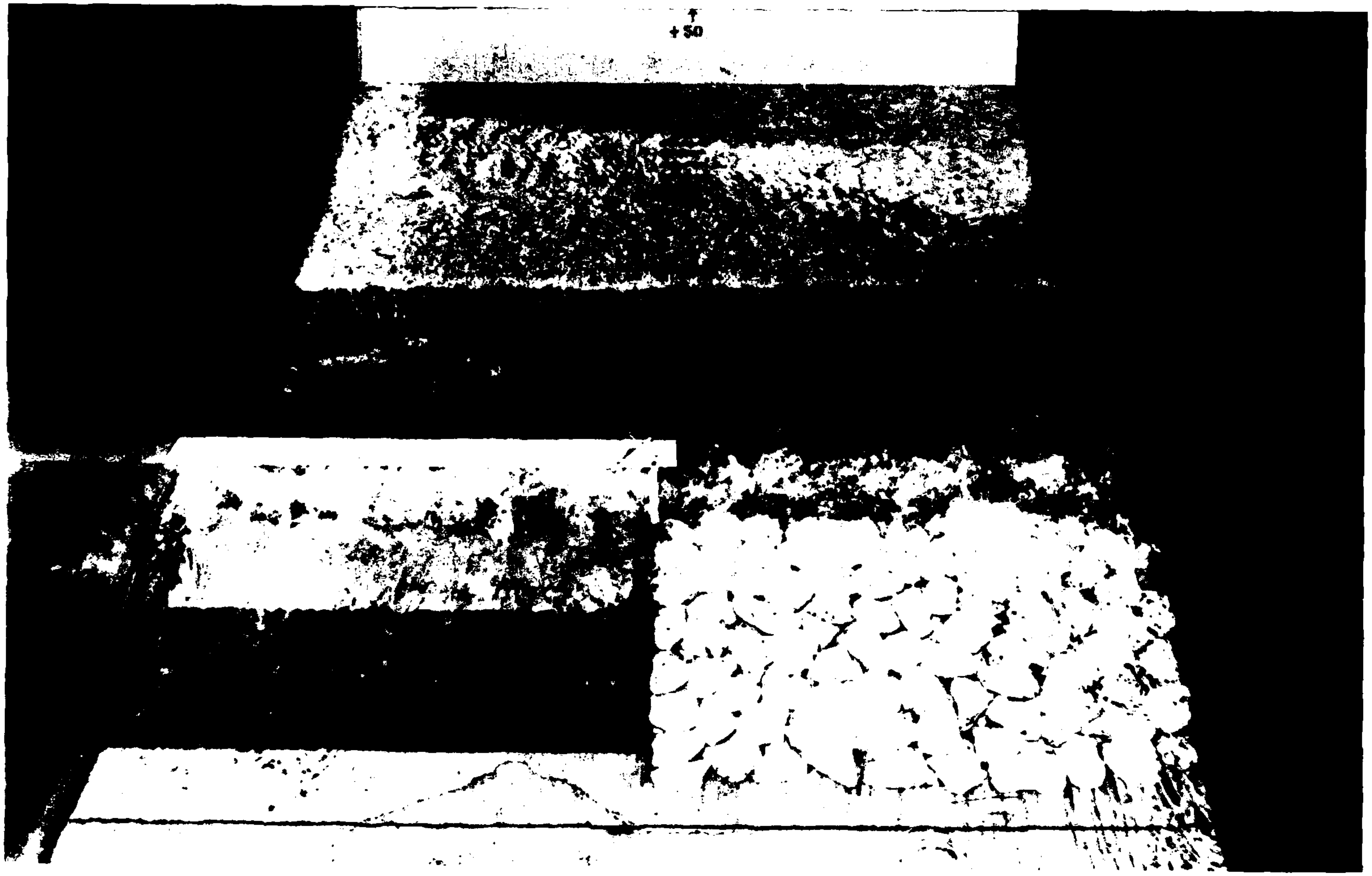

Photo 63. Sea-side view of Plan N-3-A with trestle on north slope, 8.0-sec, 12.0-ft wave at +1.9 swl

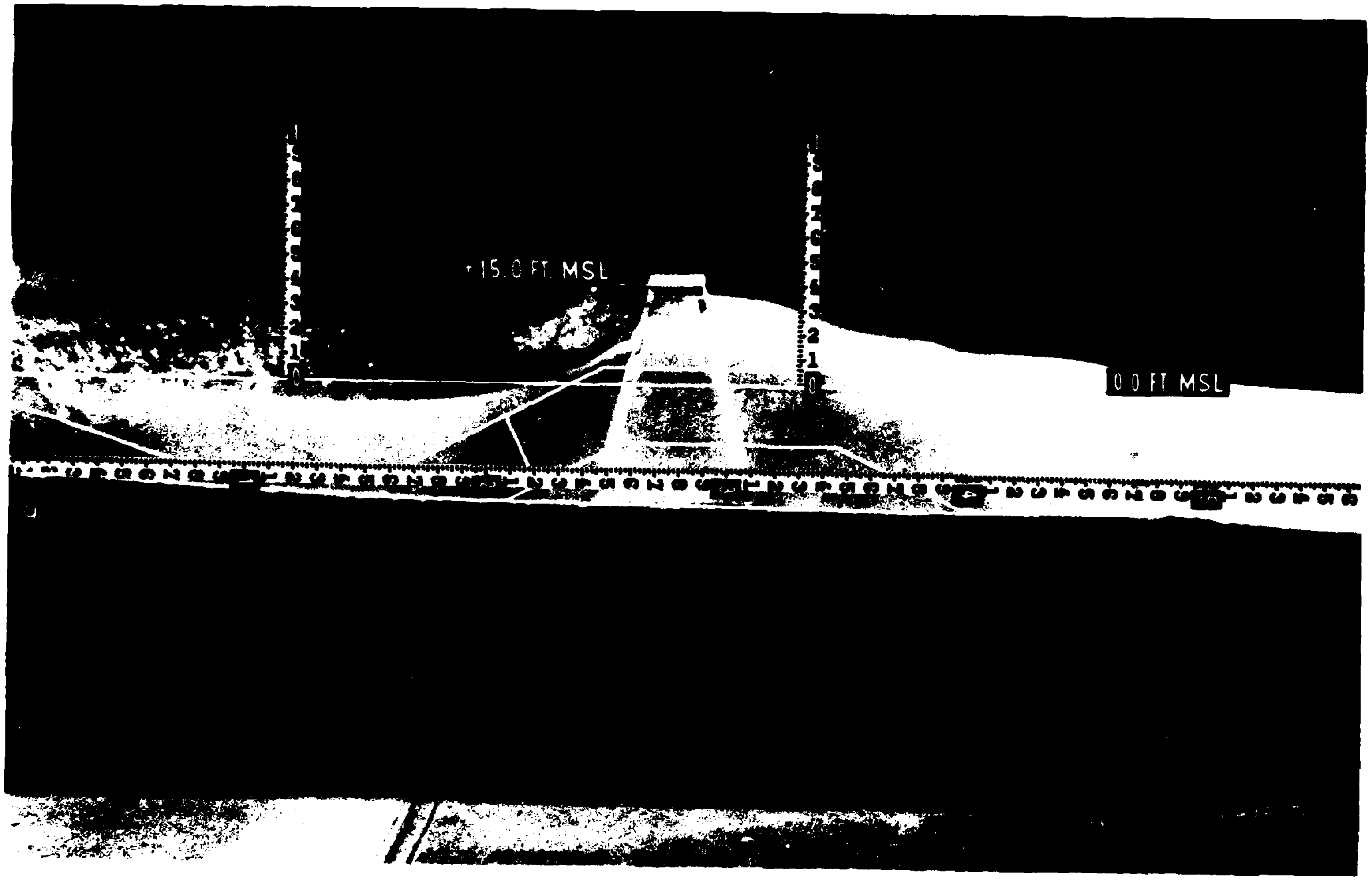

Photo 64. Side view of Plan N-3-A with trestle on north slope, 11.0-sec, 15.0-ft wave at +1.9 swl

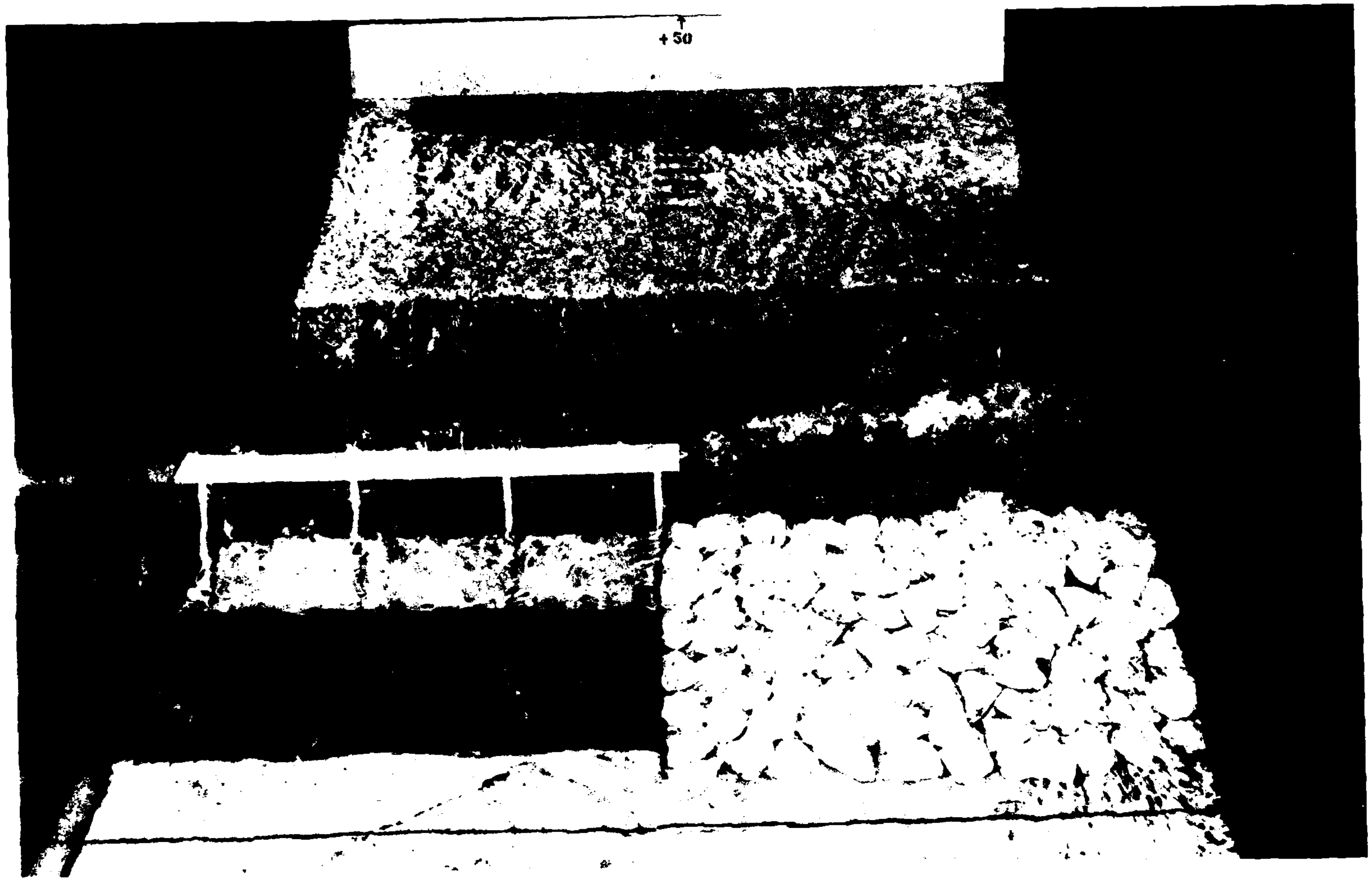

Photo 65. Sea-side view of Plan N-3-A with trestle on north slope, 11.0-sec, 15.0-ft wave at +1.9 swl

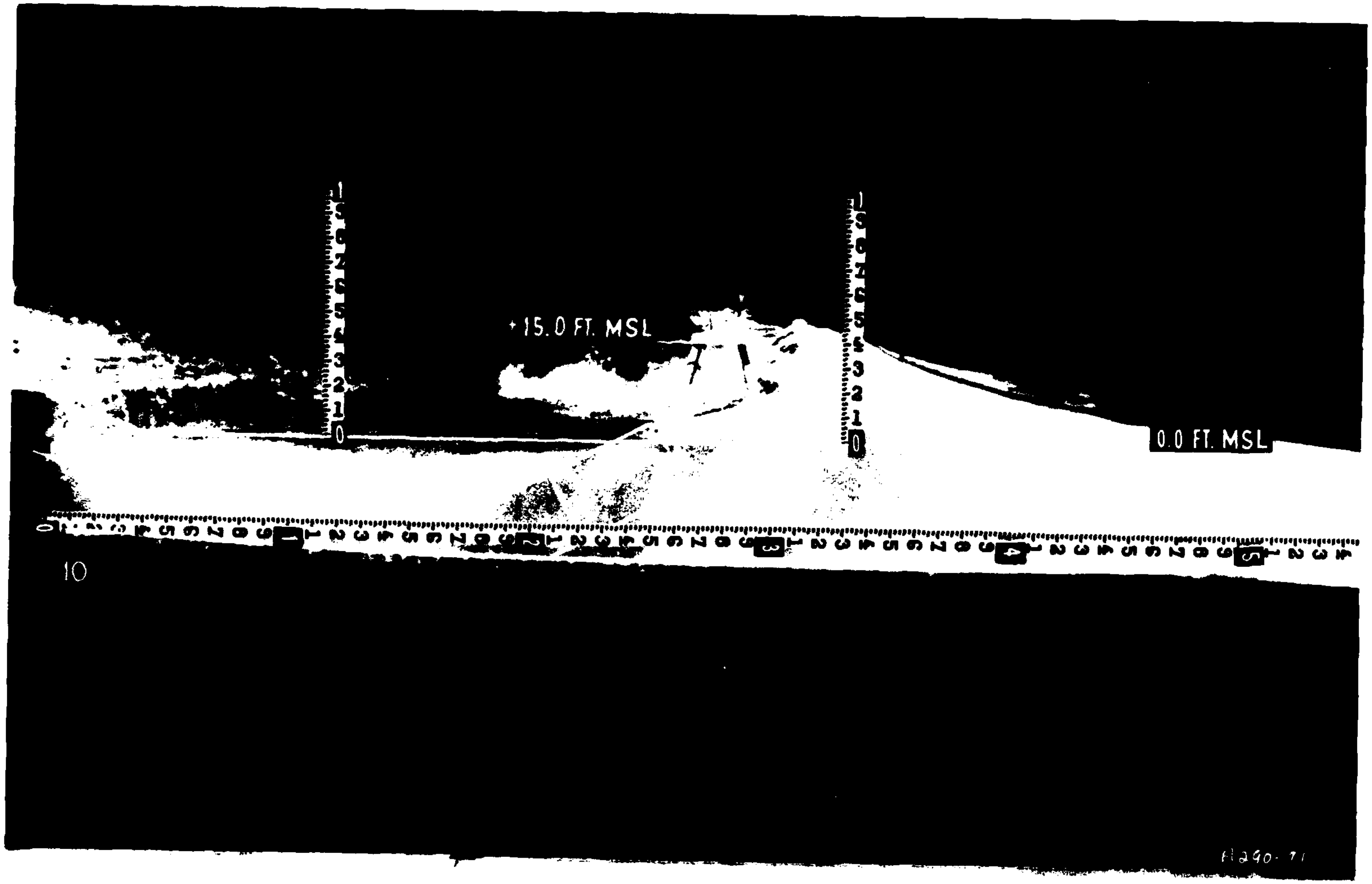

Photo 66. Side view of Plan N-3-A with trestle on north slope, 17.0-sec, 20.0-ft wave at +1.9 swl

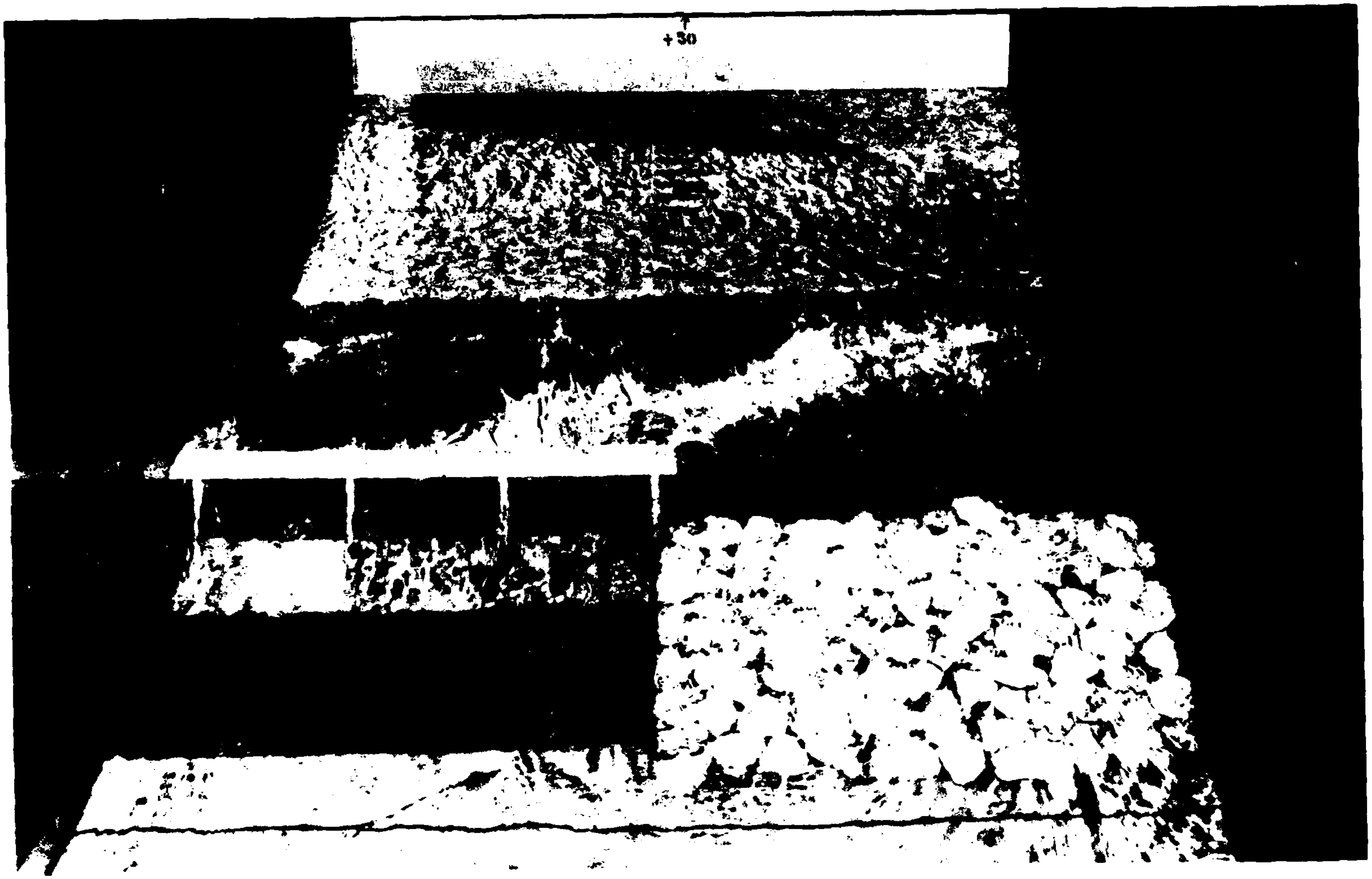

Photo 67. Sea-side view of Plan N-3-A with trestle on north slope, 17.0-sec, 20.0-ft wave at +1.9 swl

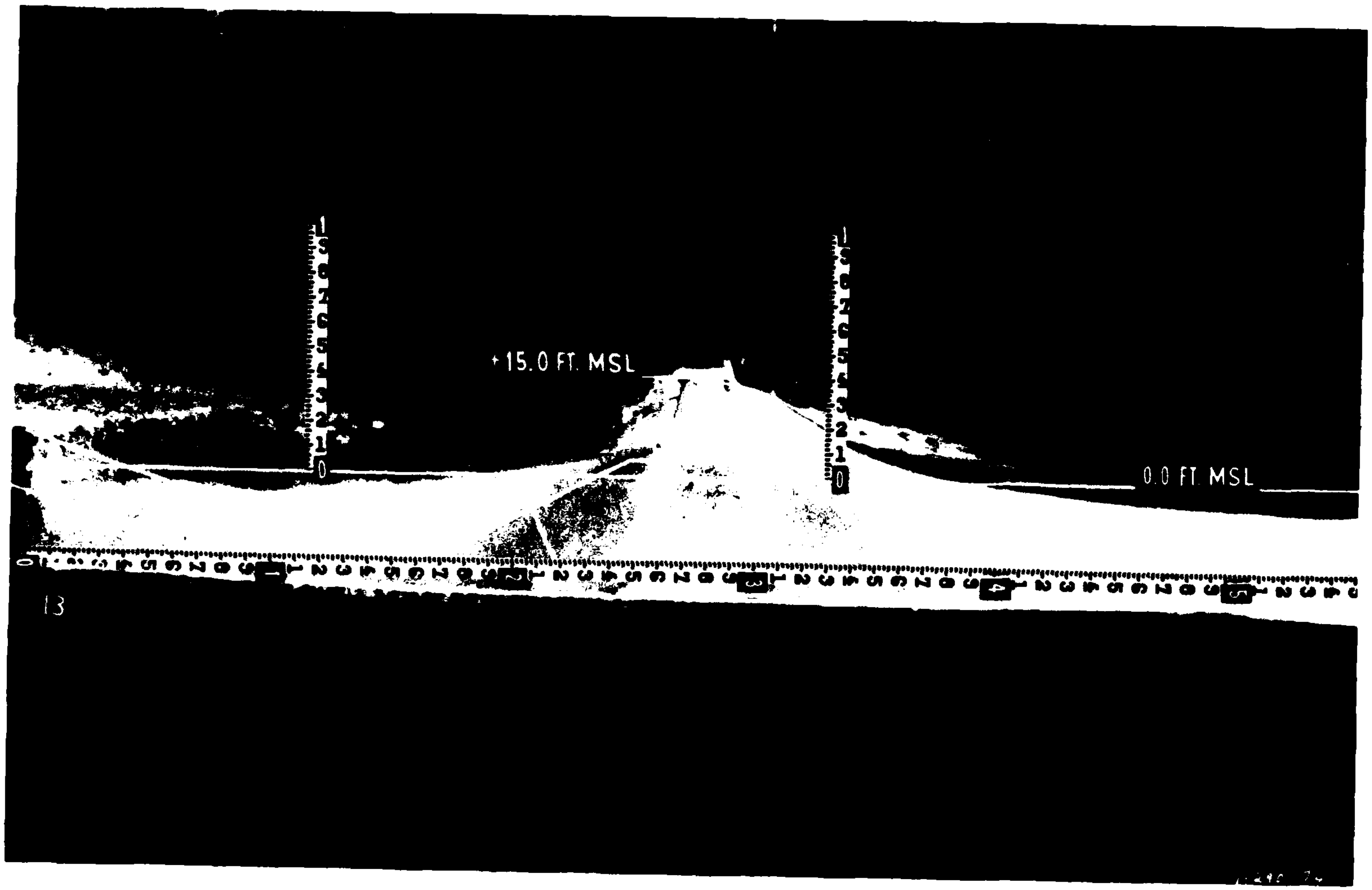

Photo 68. Side view of Plan N-3-A with trestle on north slope, 8.5-sec, 12.0-ft wave at 0.0 swl

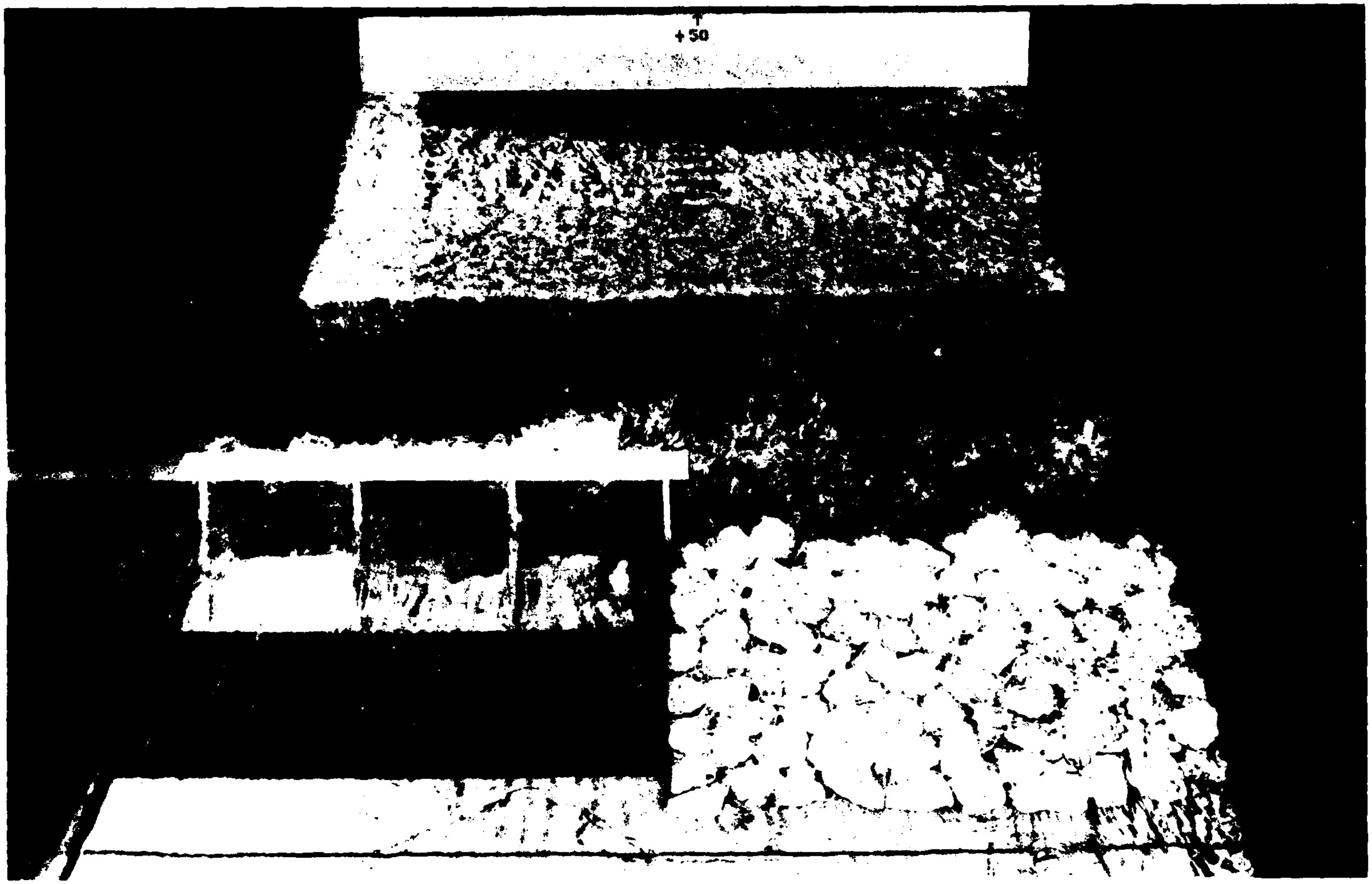

Photo 69. Sea-side view of Plan N-3-A with trestle on north slope, 8.5-sec, 12.0-ft wave at 0.0 swl

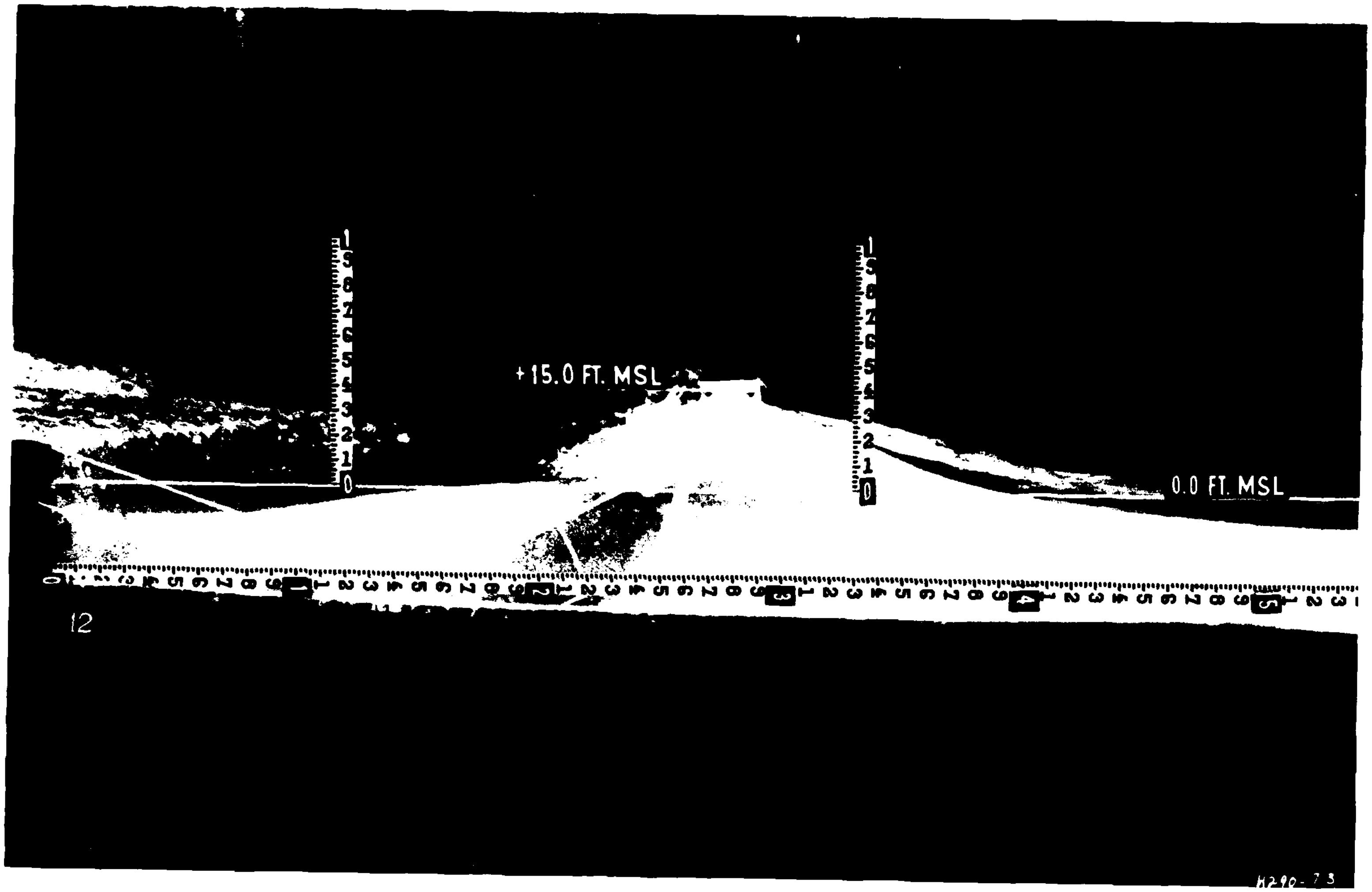

Photo 70. Side view of Plan N-3-A with trestle on north slope, 9.0-sec, 15.0-ft wave at 0.0 swl

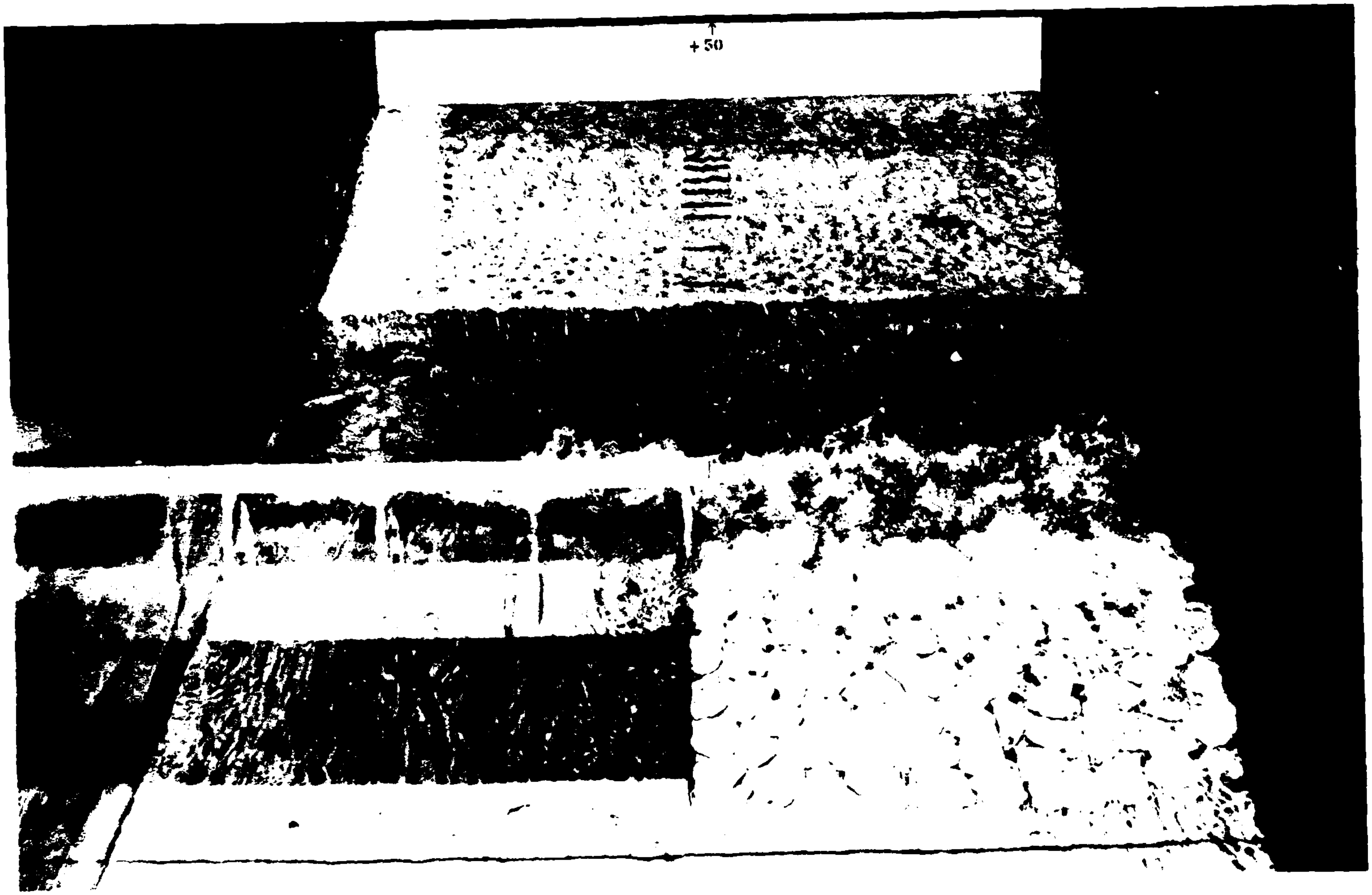

Photo 71. Sea-side view of Plan N-3-A wi[illegible] trestle on north slope, 9.0-sec, 15.0-ft wave at [illegible].0 swl

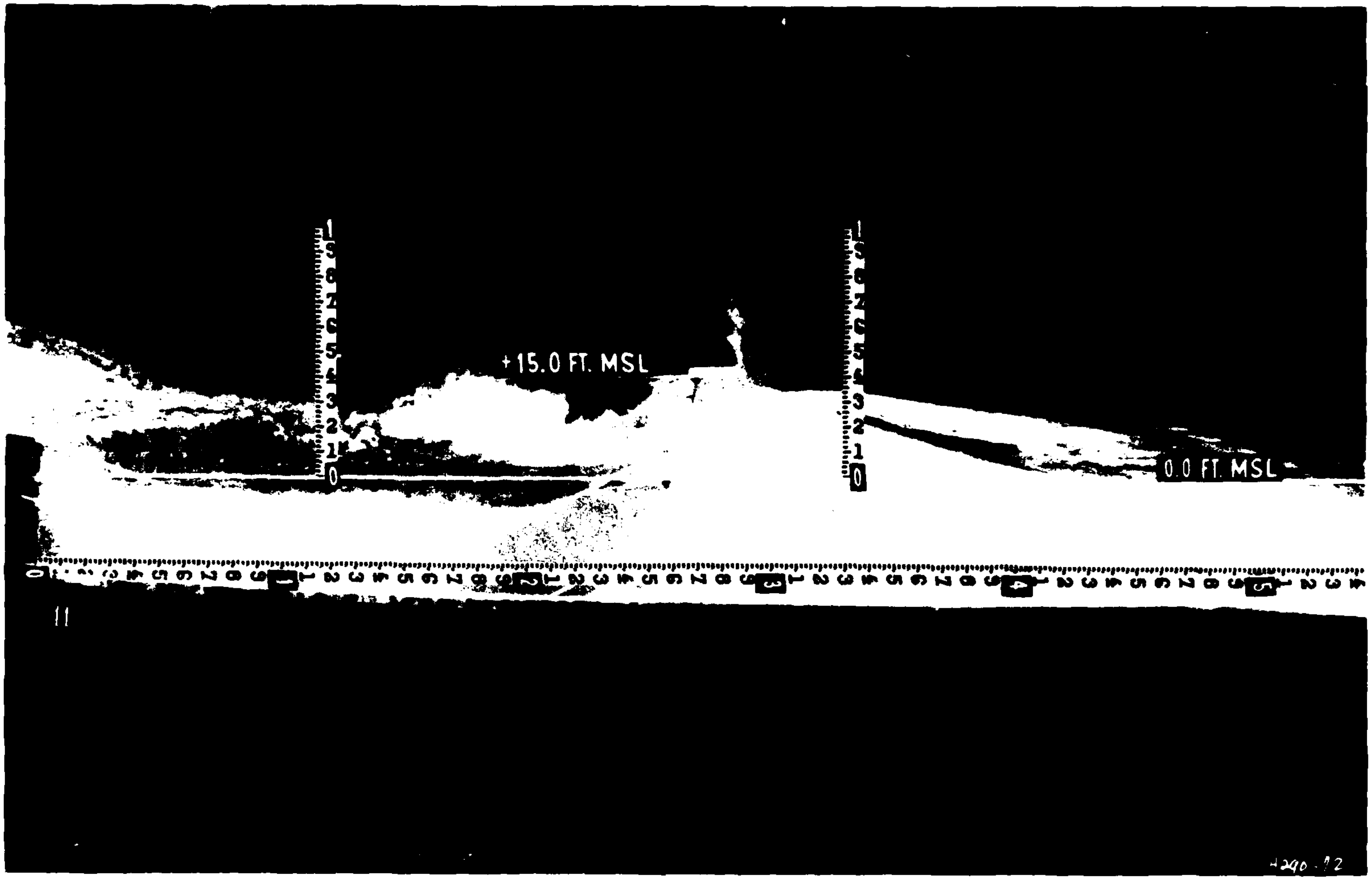

Photo 72. Side view of Plan N-3-A with trestle on north slope, 17.0-sec, 20.0-ft wave at 0.0 swl

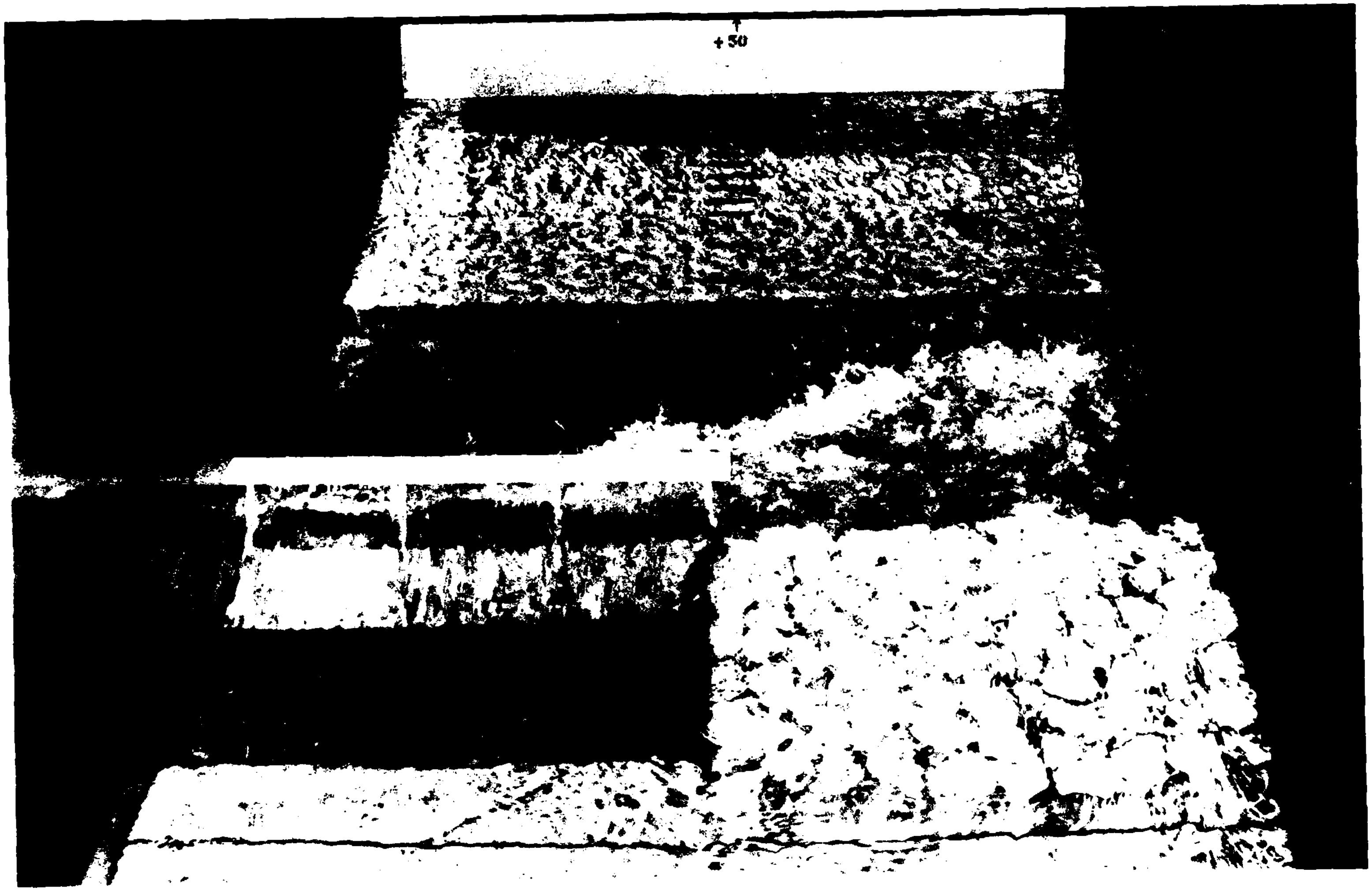

Photo 73. Sea-side view of Plan N-3-A with trestle on north slope, 17.0-sec, 20.0-ft wave at 0.0 swl

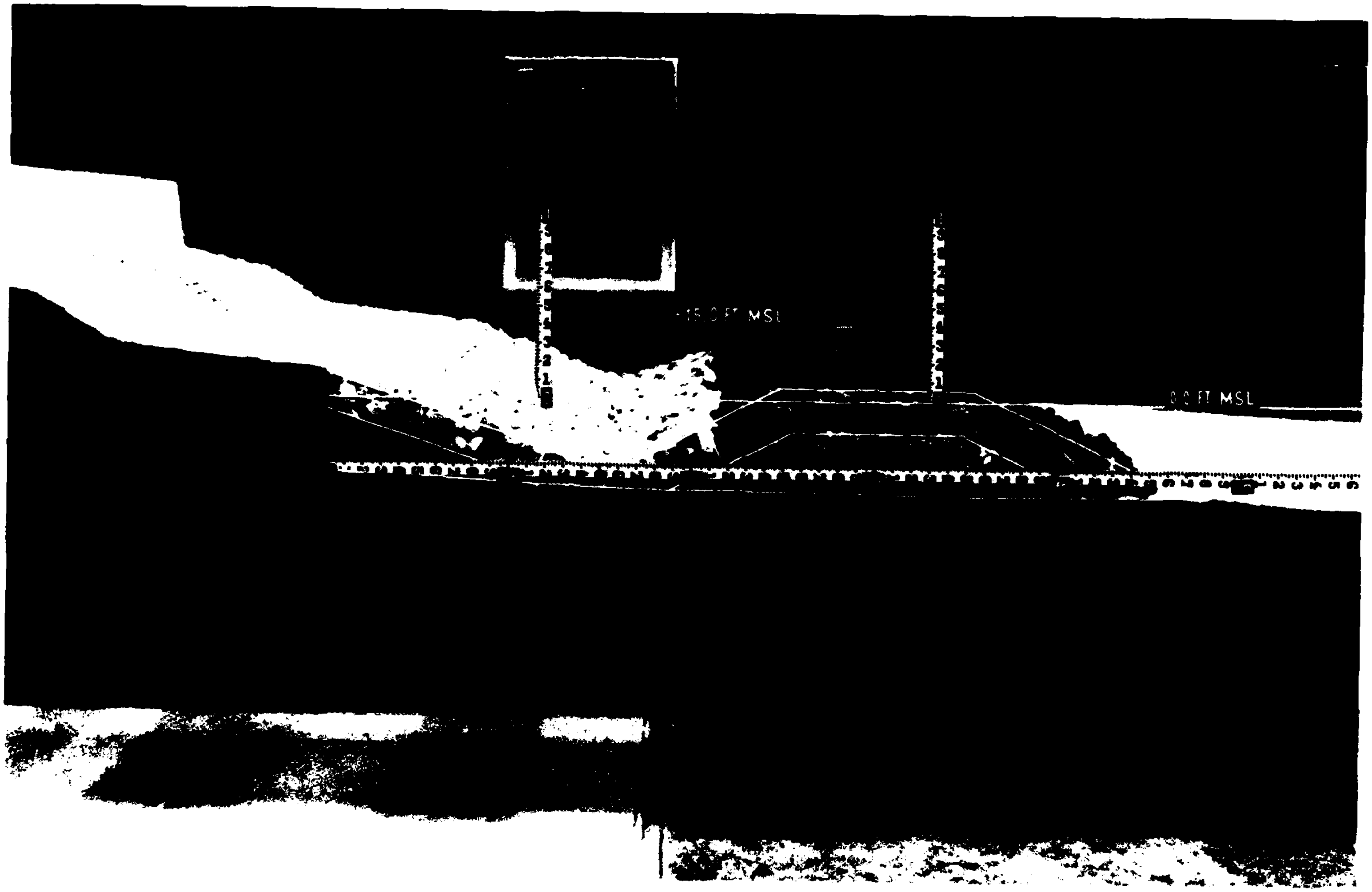

Photo 74. Side view of Plan N-5 before testing

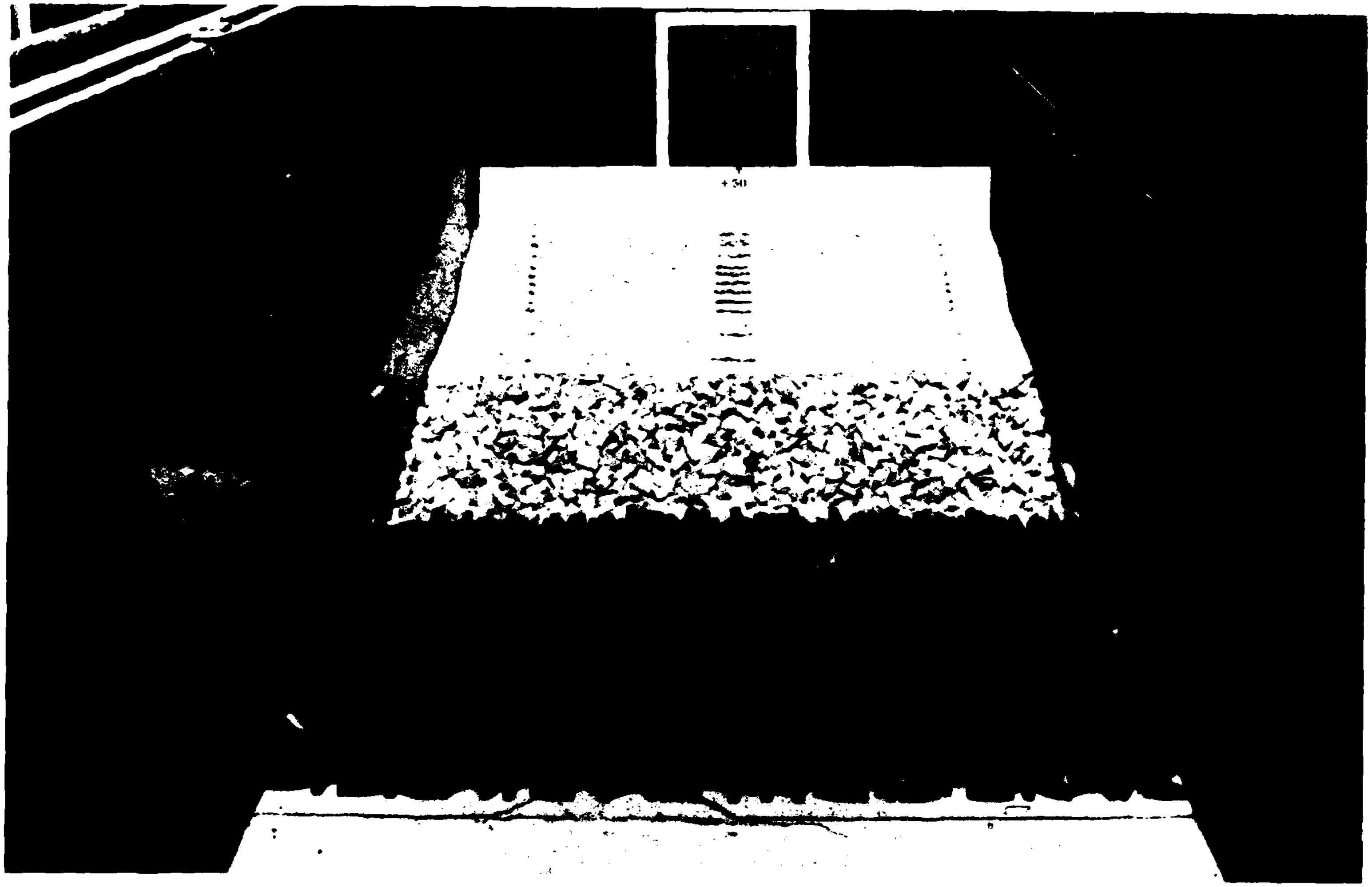

Photo 75. Sea-side view of Plan N-5 before testing

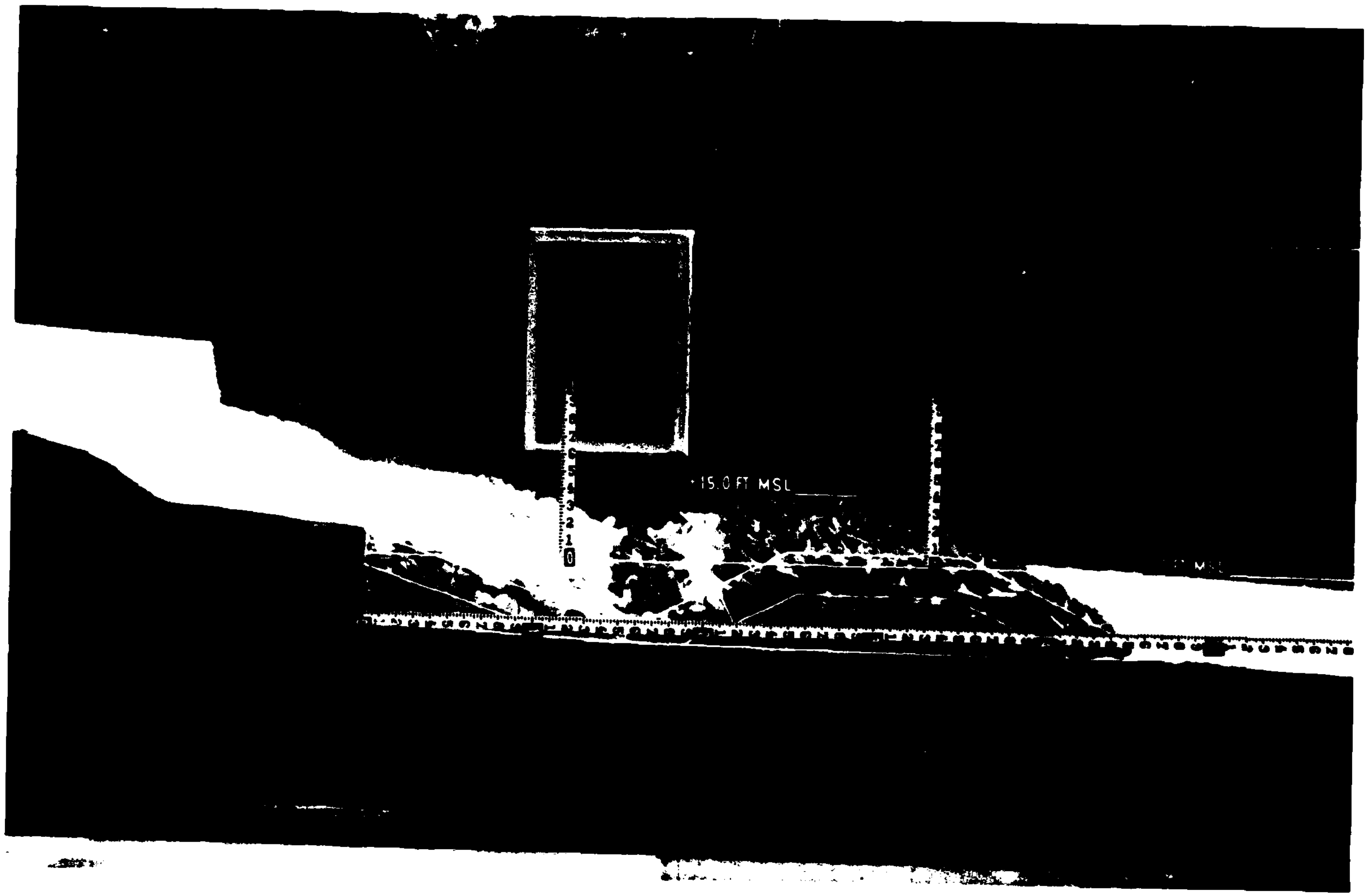

Photo 76. Side view of Plan N-5 after testing Hydrograph A (Plate 1 and Table 1)

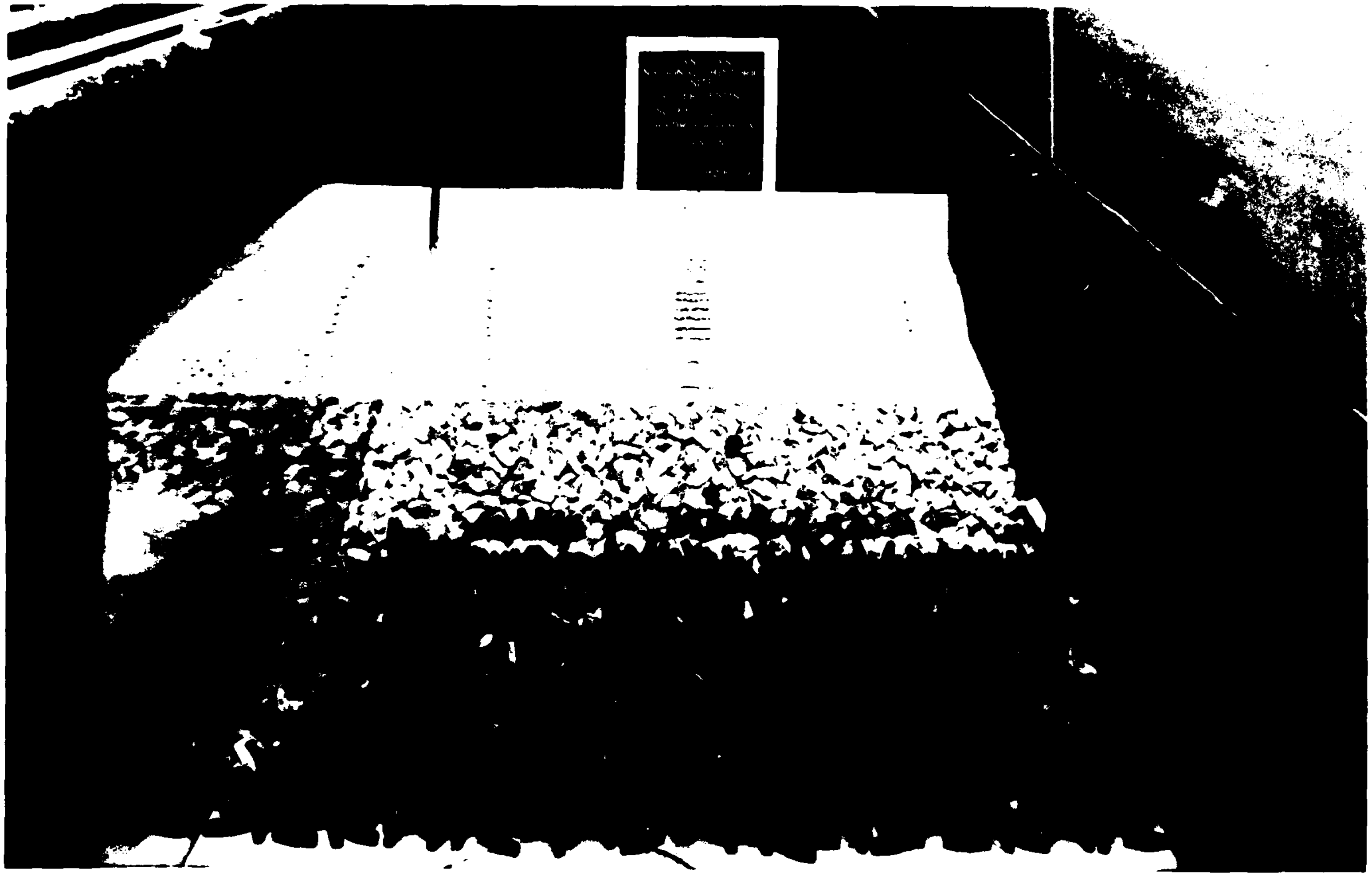

Photo 77. Sea-side view of Plan N-5 after testing Hydrograph A (Plate 1 and Table 1)

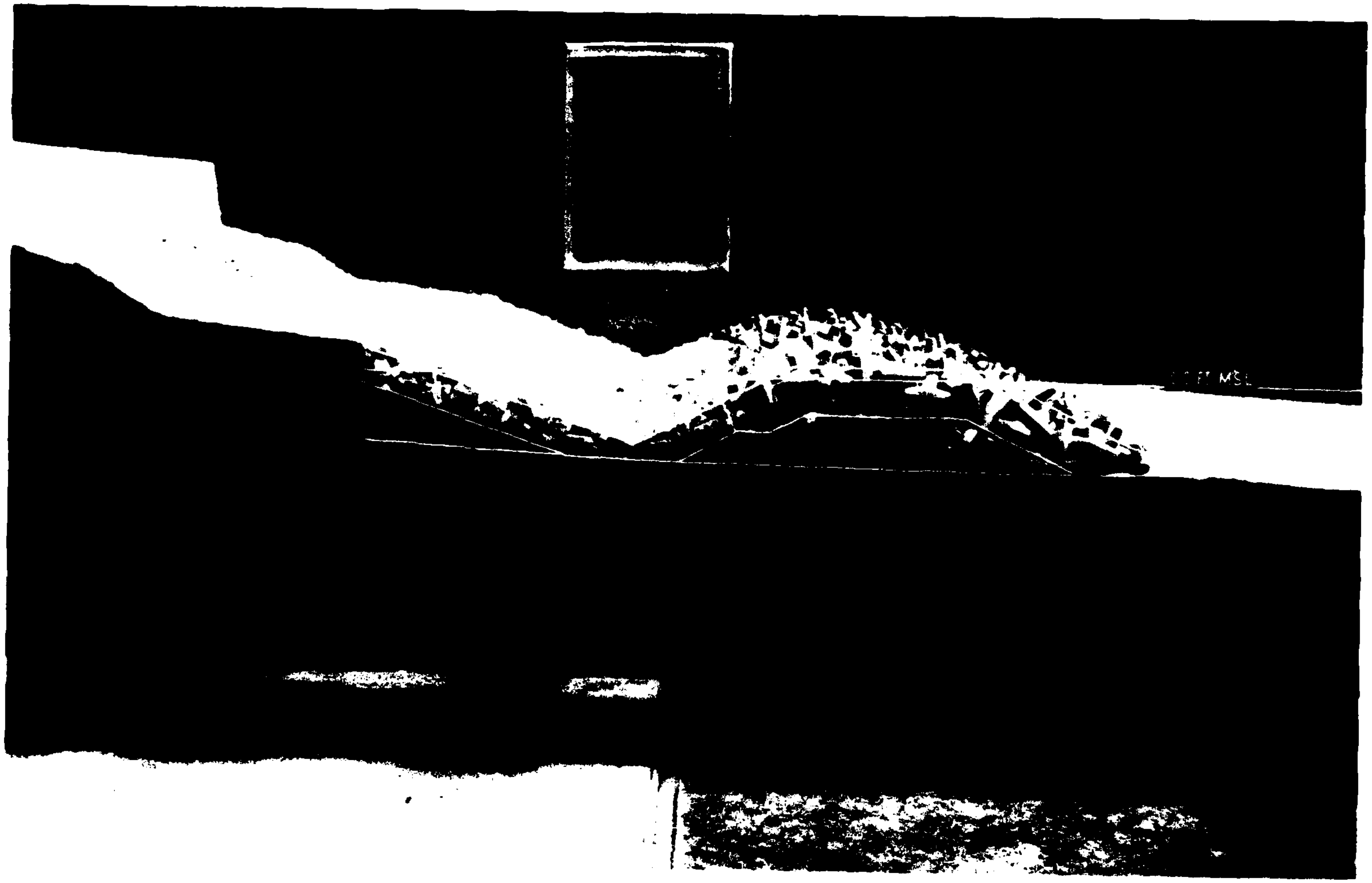

Photo 78. Side view of Plan N-6 before testing, 1st test section

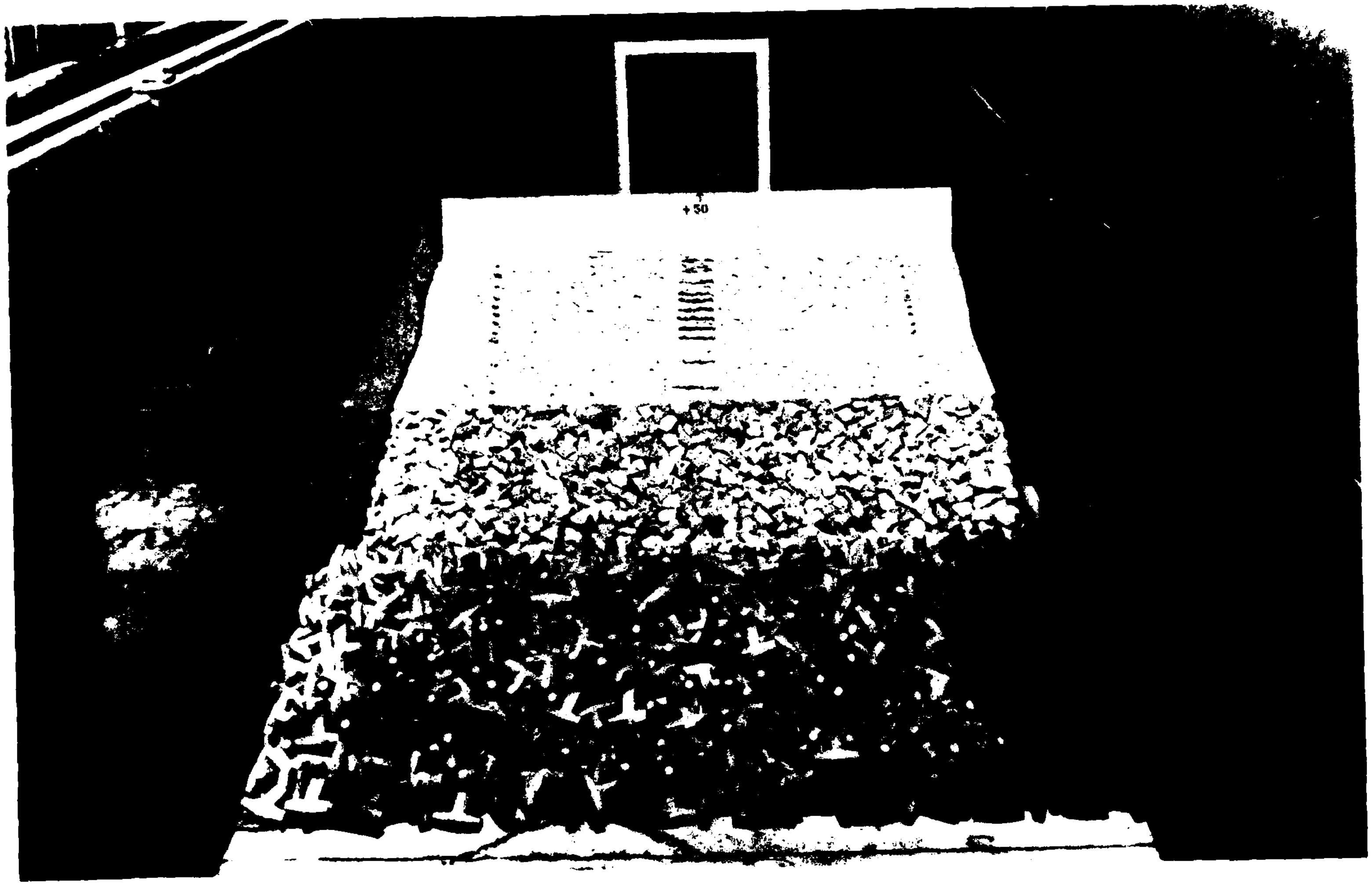

Photo 79. Sea-side view of Plan N-6 before testing, 1st test section

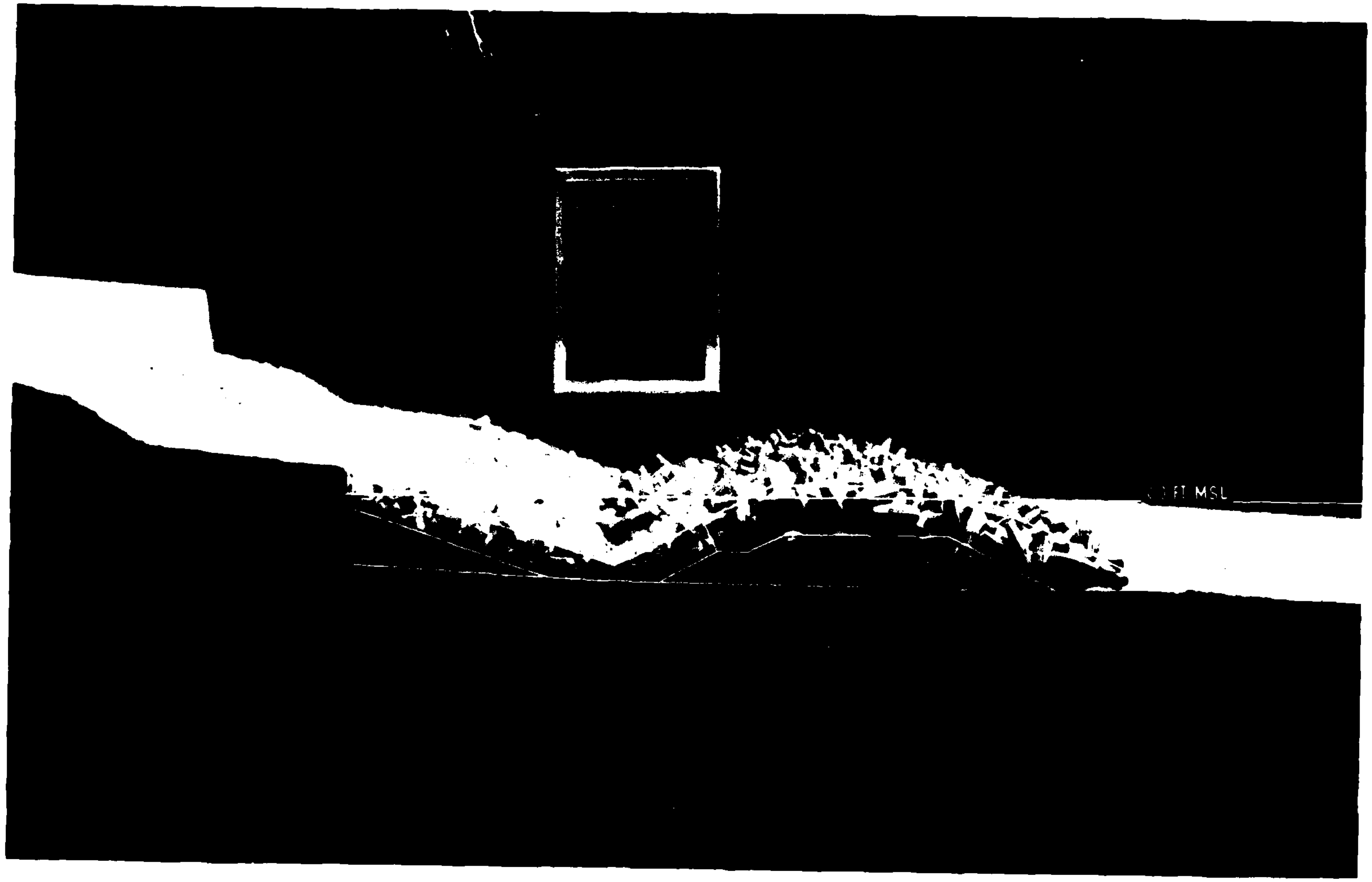

Photo 80. Side view of Plan N-6 after testing Hydrograph A (Plate 1 and Table 1), 1st test section

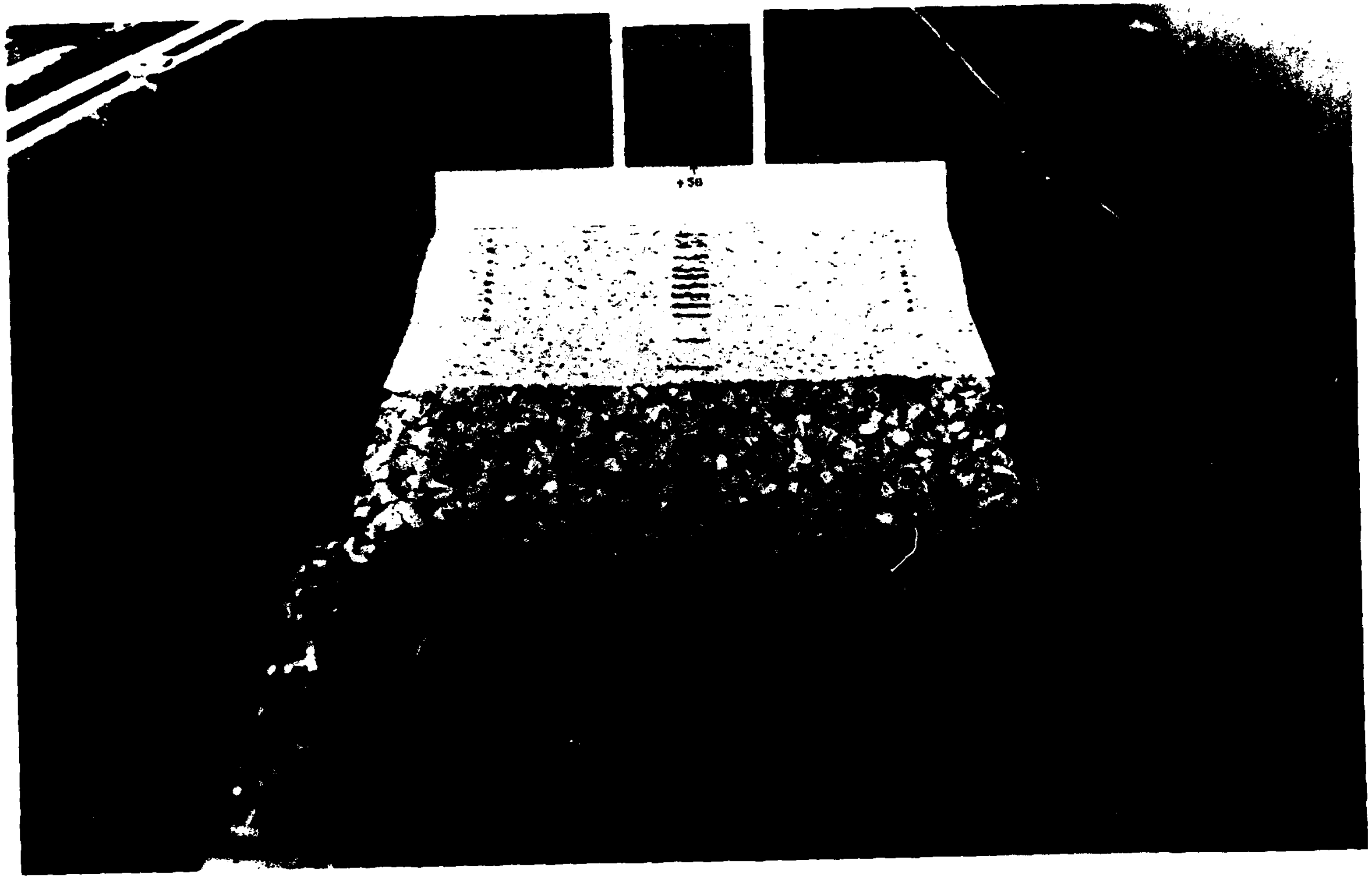

Photo 81. Sea-side view of Plan N-6 after testing Hydrograph A (Plate 1 and Table 1), 1st test section

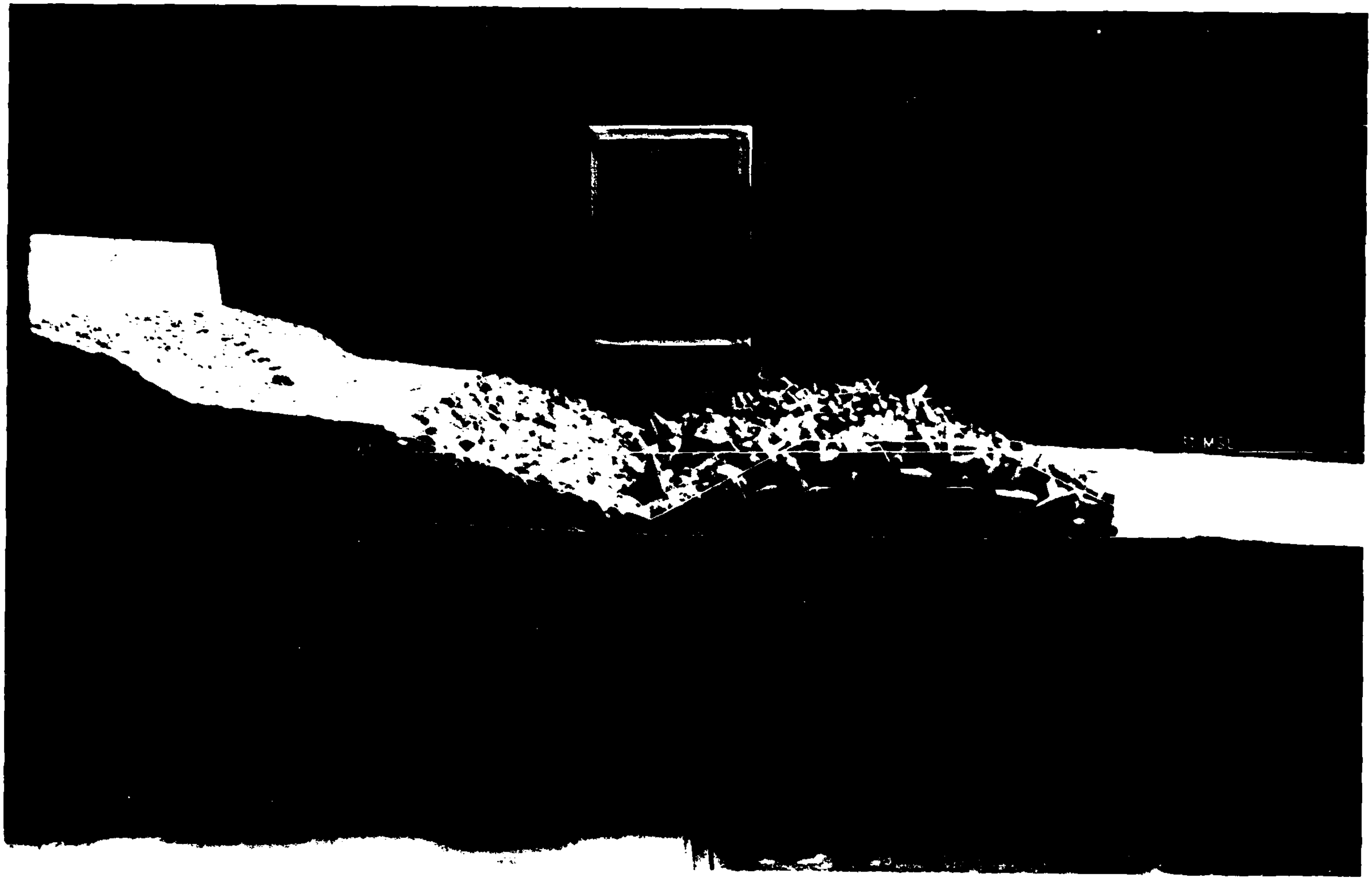

Photo 82. Side view of Plan N-6 after testing Hydrograph A (Plate 1 and Table 1), 2nd test section

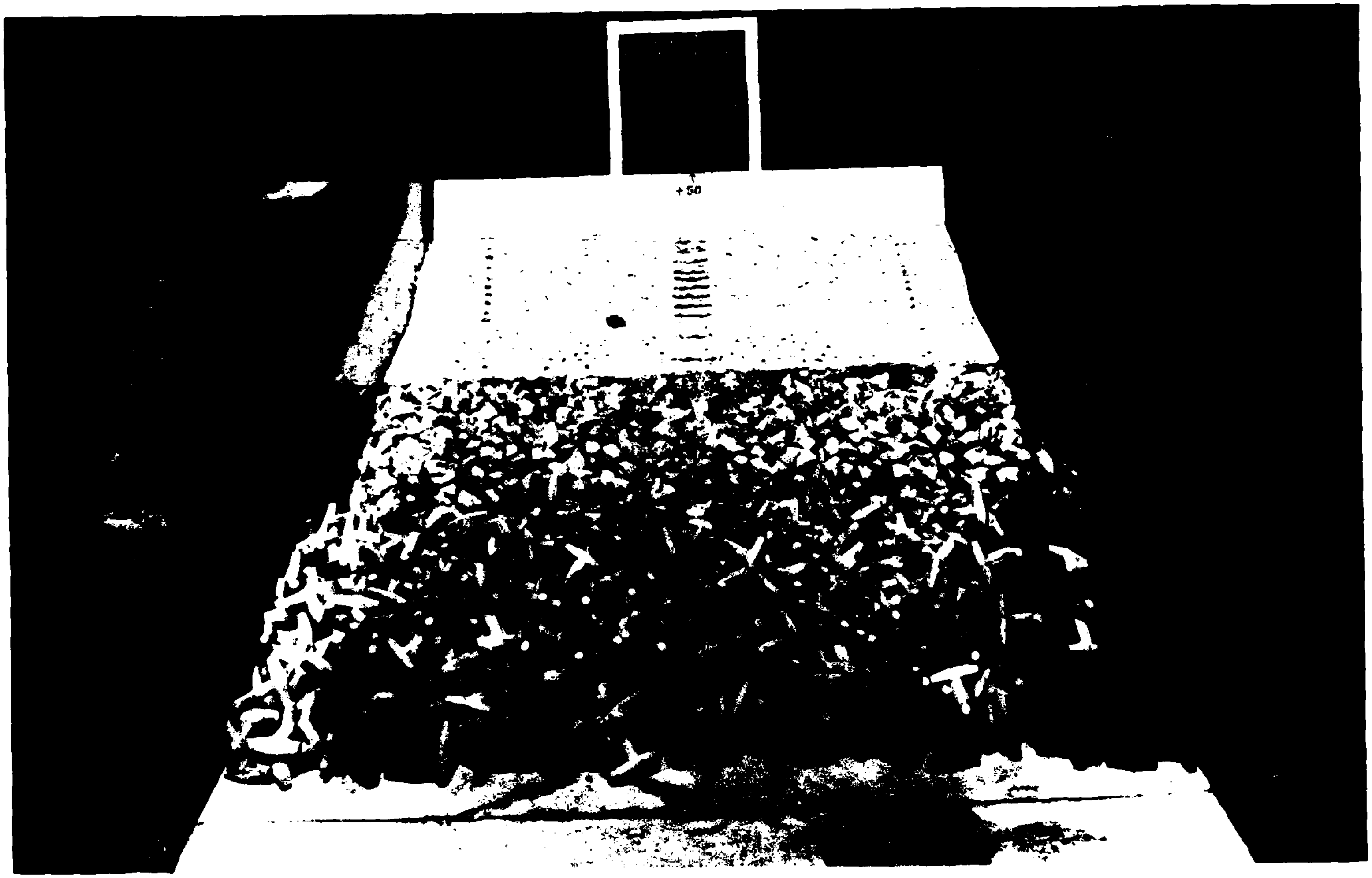

Photo 83. Sea-side view of Plan N-6 after testing Hydrograph A (Plate 1 and Table 1), 2nd test section

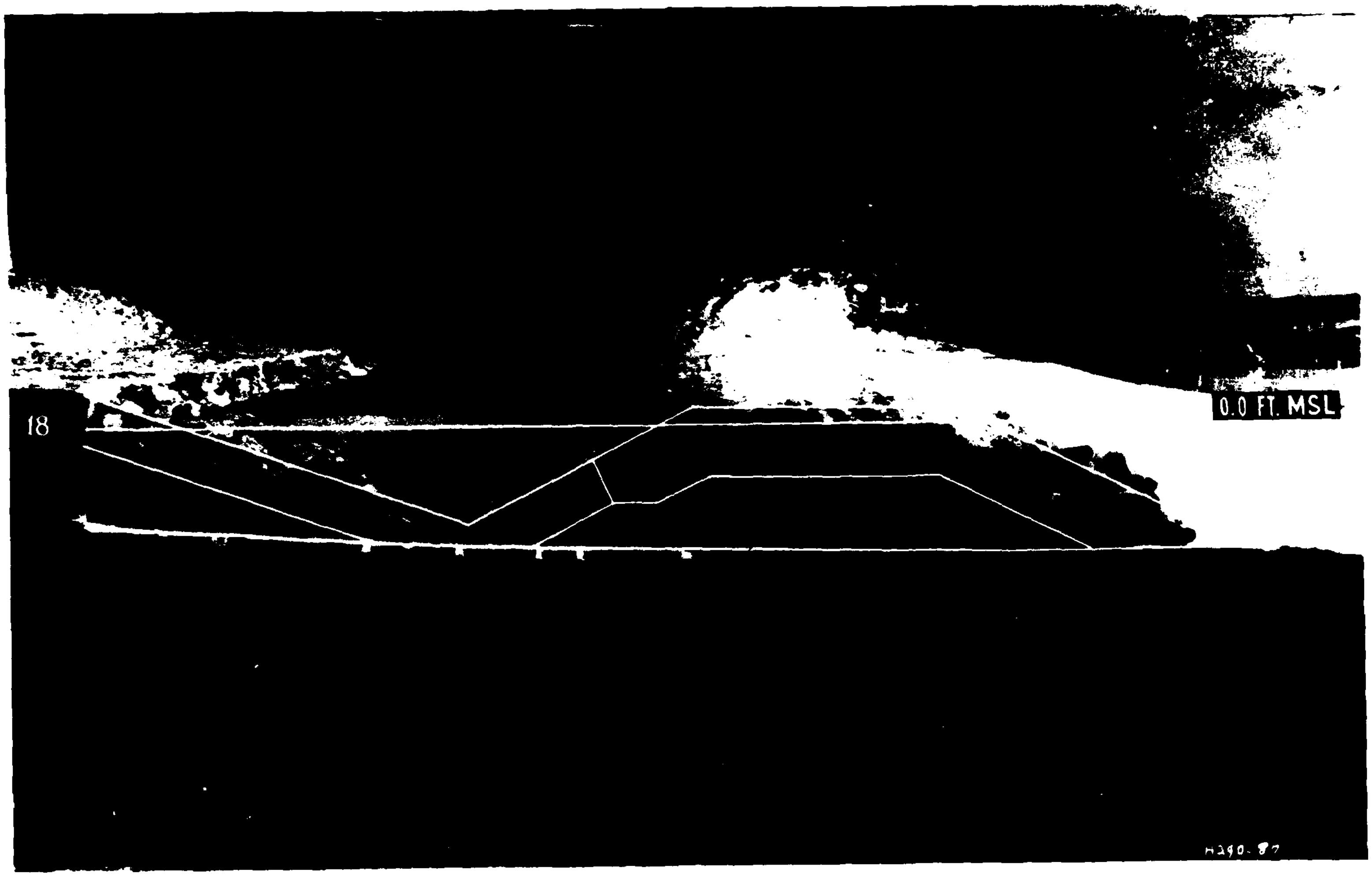

Photo 84. Side view of Plan N-6, 15.0-sec, 20.0-ft breaking wave at 0.0 swl, Step 1 of Hydrograph A (Plate 1 and Table 1)

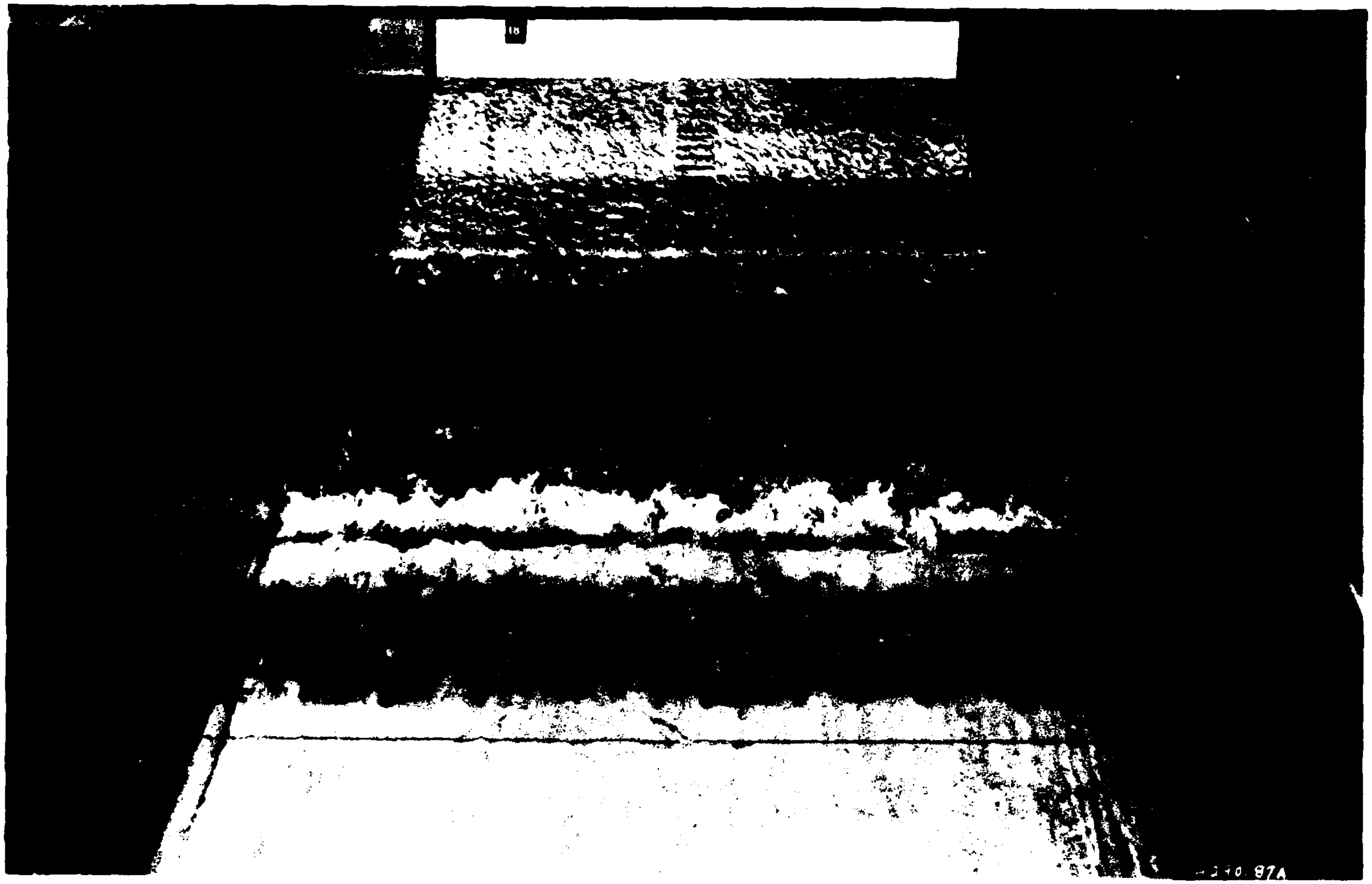

Photo 85. Sea-side view of Plan N-6, 15.0-sec, 20.0-ft breaking wave at 0.0 swl, Step 1 of Hydrograph A (Plate 1 and Table 1)

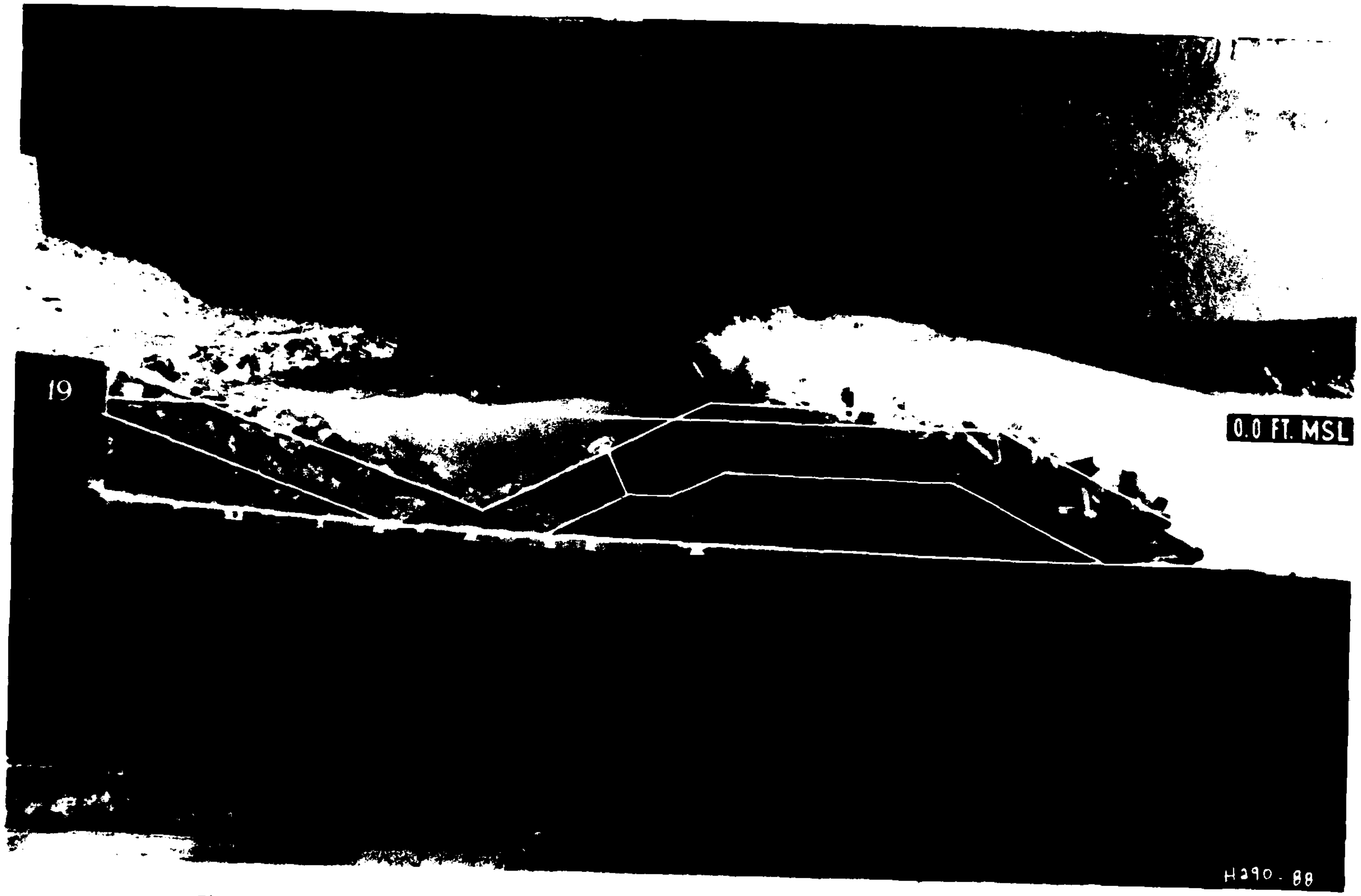

Photo 86. Side view of Plan N-6, 17.0-sec, 22.7-ft breaking wave at 0.0 swl, Step 2 of Hydrograph A (Plate 1 and Table 1)

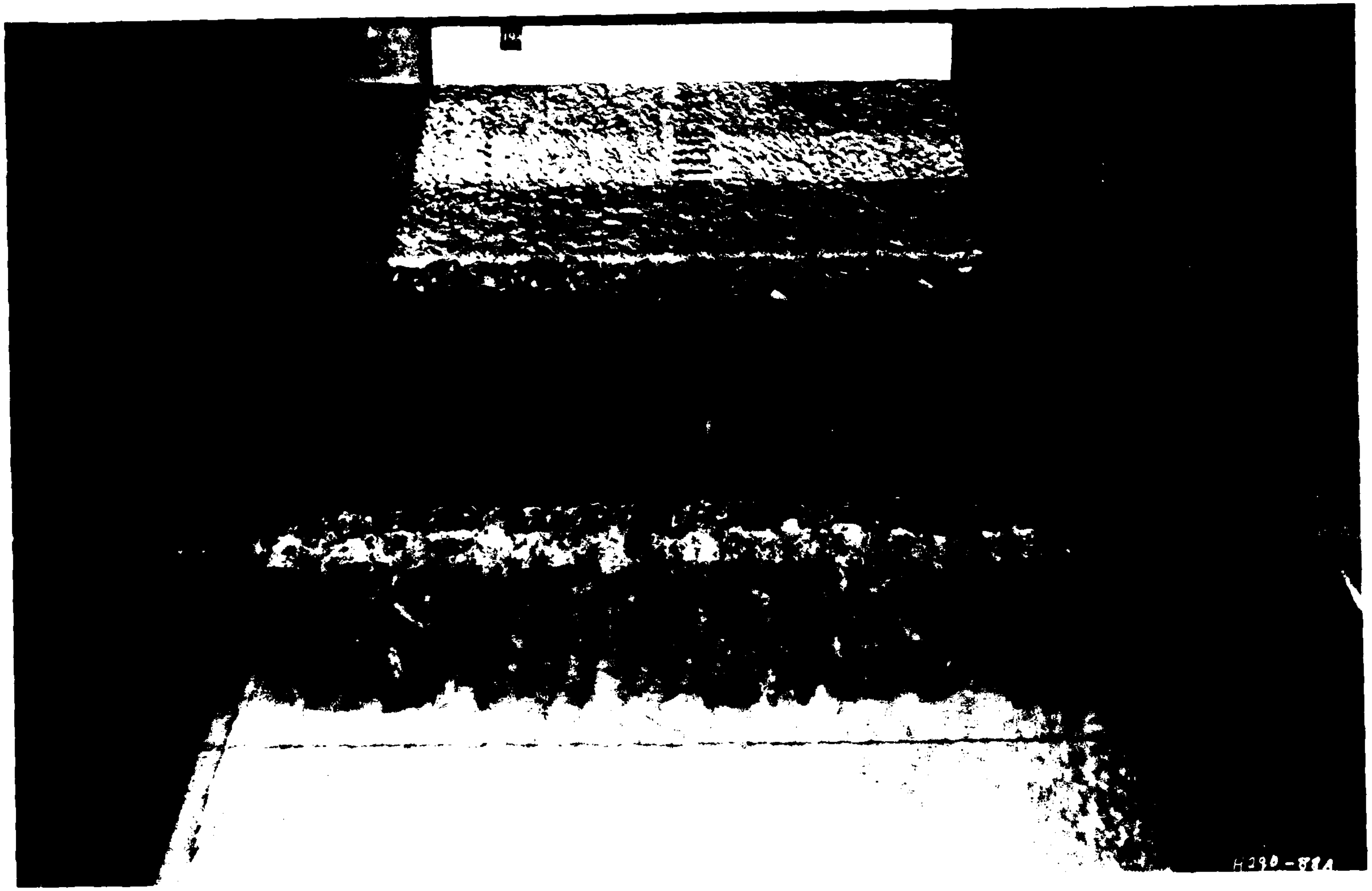

Photo 87. Sea-side view of Plan N-6, 17.0-sec, 22.7-ft breaking wave at 0.0 swl, Step 2 of Hydrograph A (Plate 1 and Table 1)

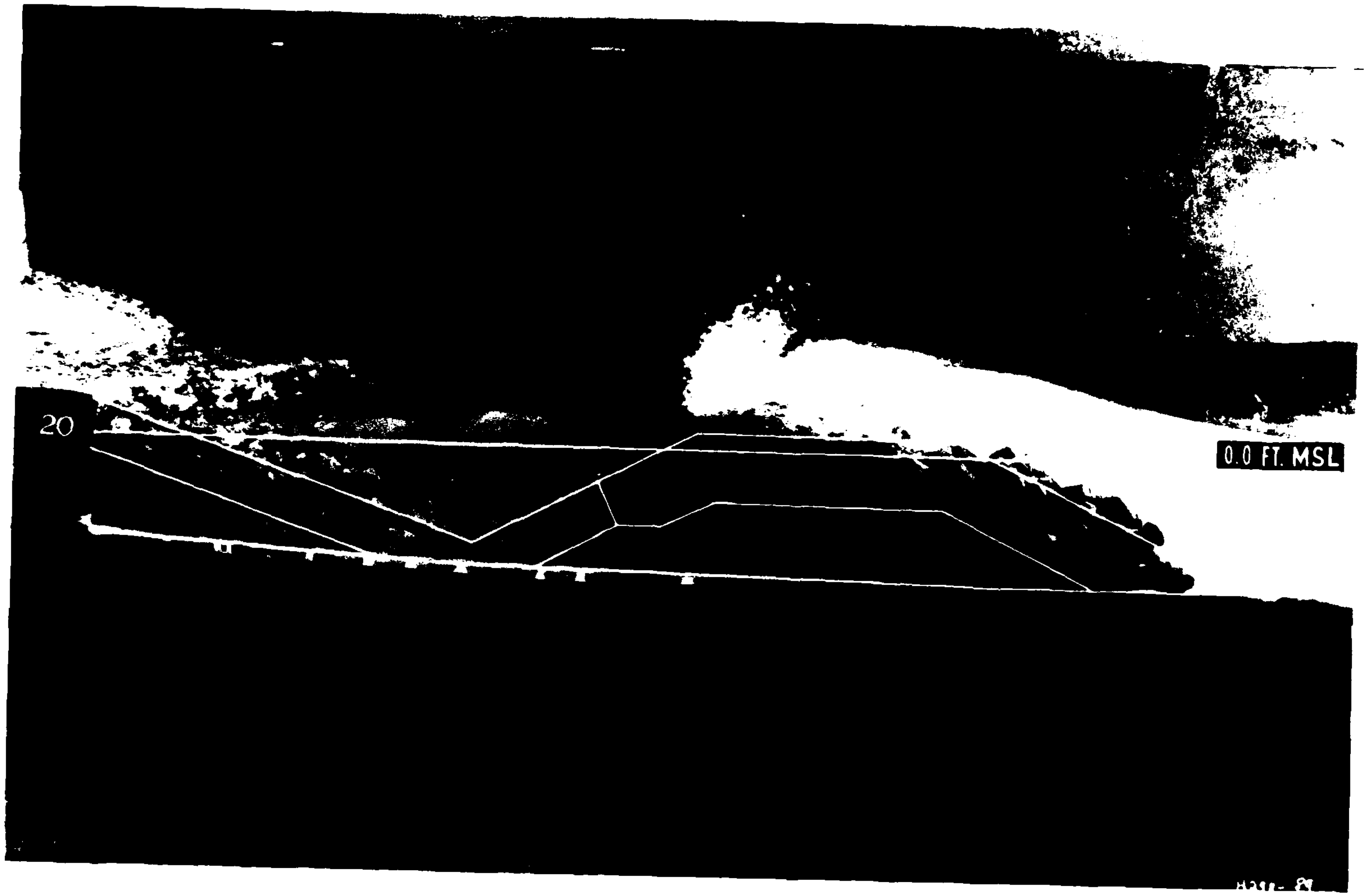

Photo 88. Side view of Plan N-6, 15.0-sec, 21.2-ft breaking wave at +1.9 swl, Step 3 of Hydrograph A (Plate 1 and Table 1)

Photo 89. Sea-side view of Plan N-6, 15.0-sec, 21.2-ft breaking wave at +1.9 swl, Step 3 of Hydrograph A (Plate 1 and Table 1)

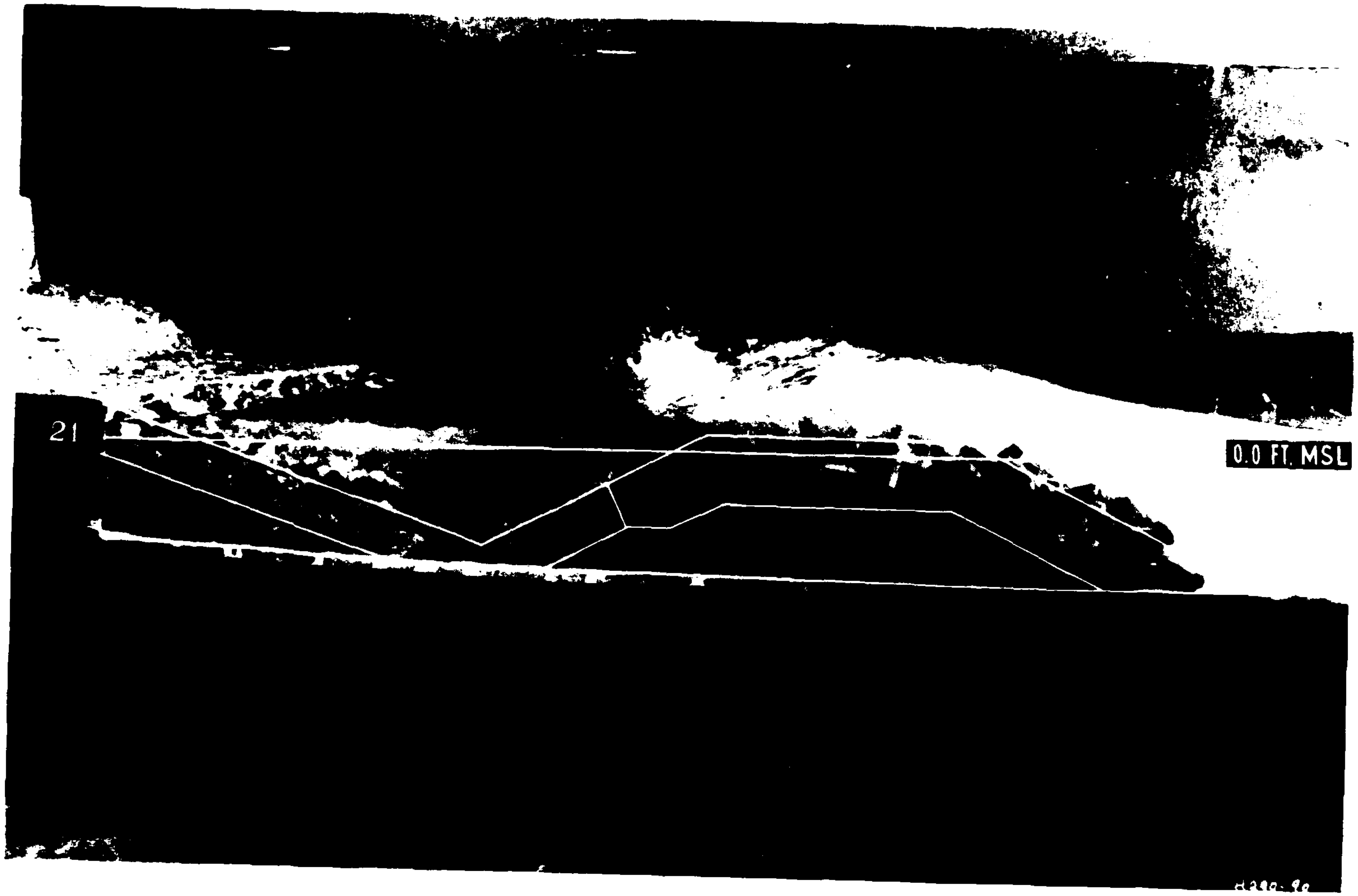

Photo 90. Side view of Plan N-6, 17.0-sec, 23.3-ft breaking wave at +1.9 swl, Step 4 of Hydrograph A (Plate 1 and Table 1)

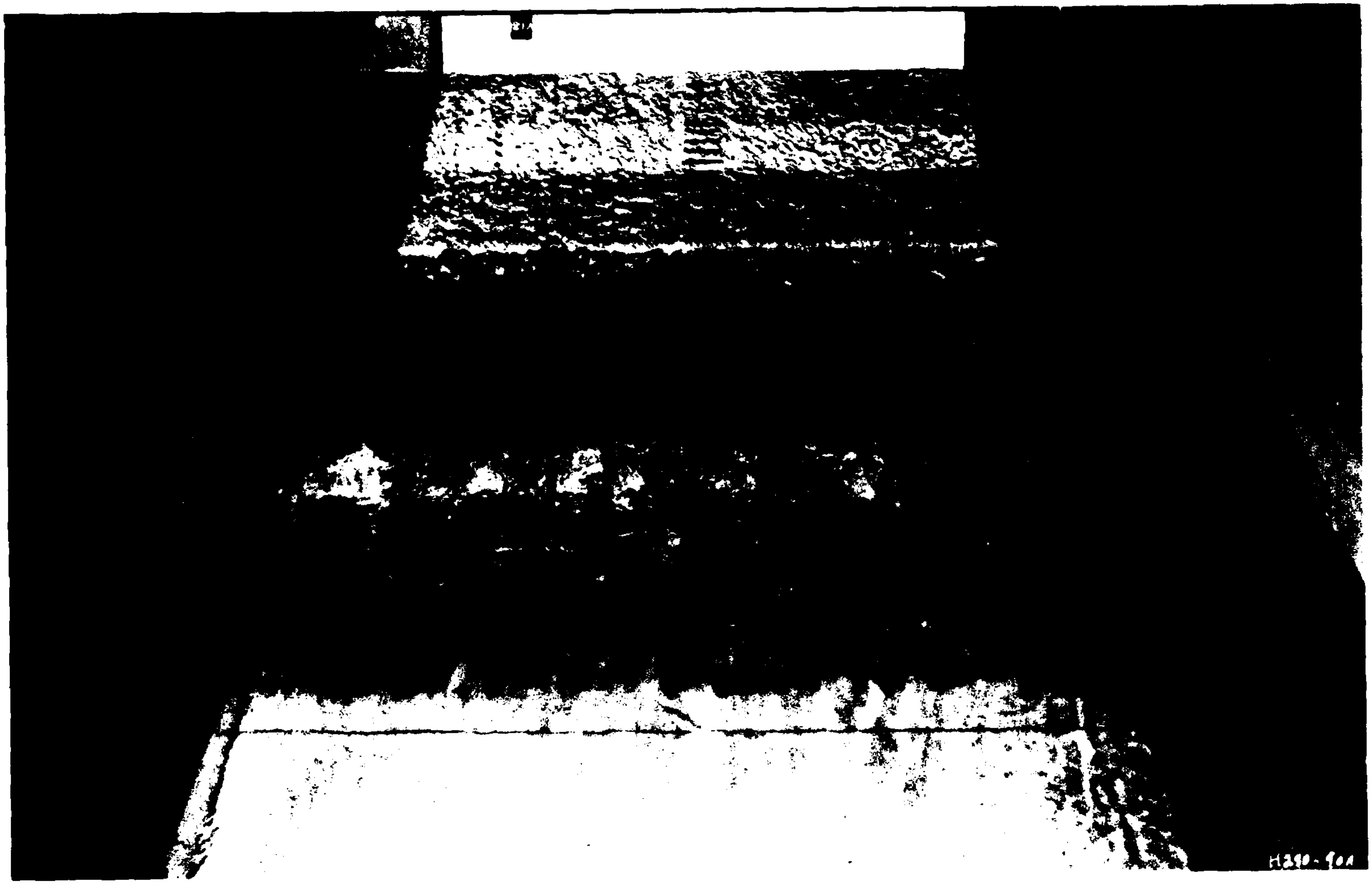

Photo 91. Sea-side view of Plan N-6, 17.0-sec, 23.3-ft breaking wave at +1.9 swl, Step 4 of Hydrograph A (Plate 1 and Table 1)

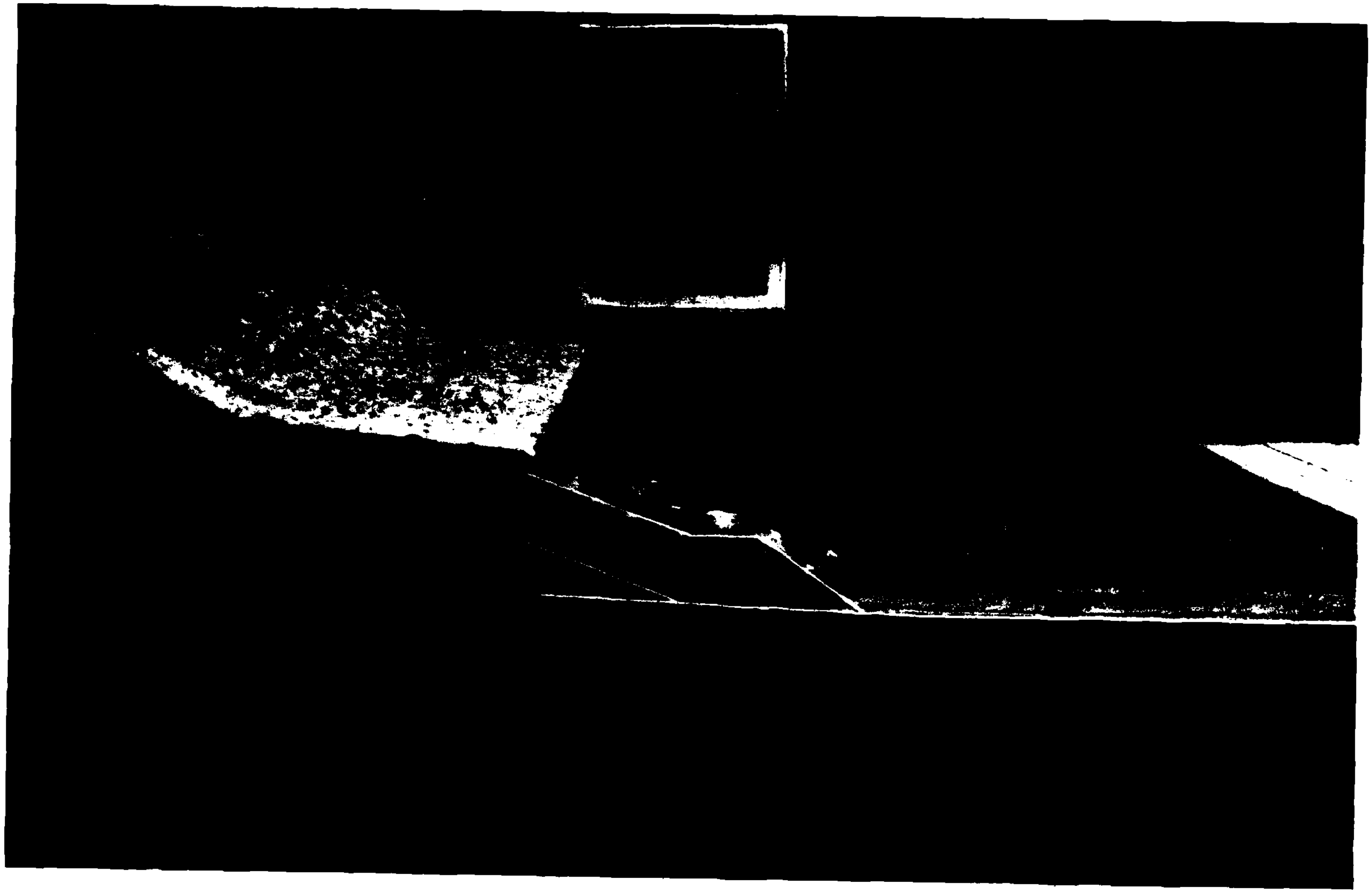

Photo 92. Side view of Plan N-7 after testing 3.0 hr of 15.0-sec, 16.5-ft breaking wave at +1.9 swl

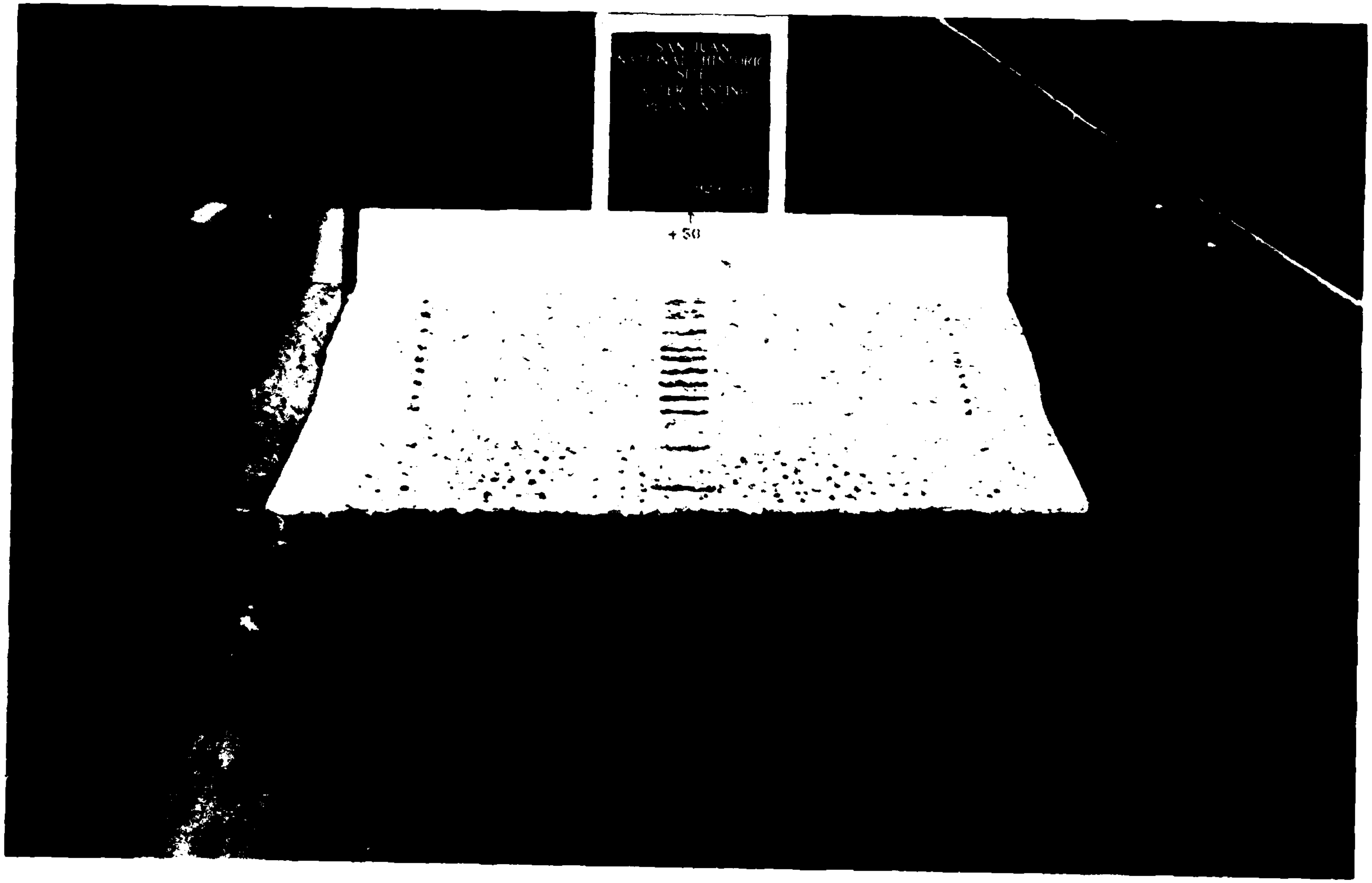

Photo 93. Sea-side view of Plan N-7 after testing 3.0 hr of 15.0-sec, 16.5-ft breaking wave at +1.9 swl

Photo 94. Side view of Plan W-1 before testing, 1st test section

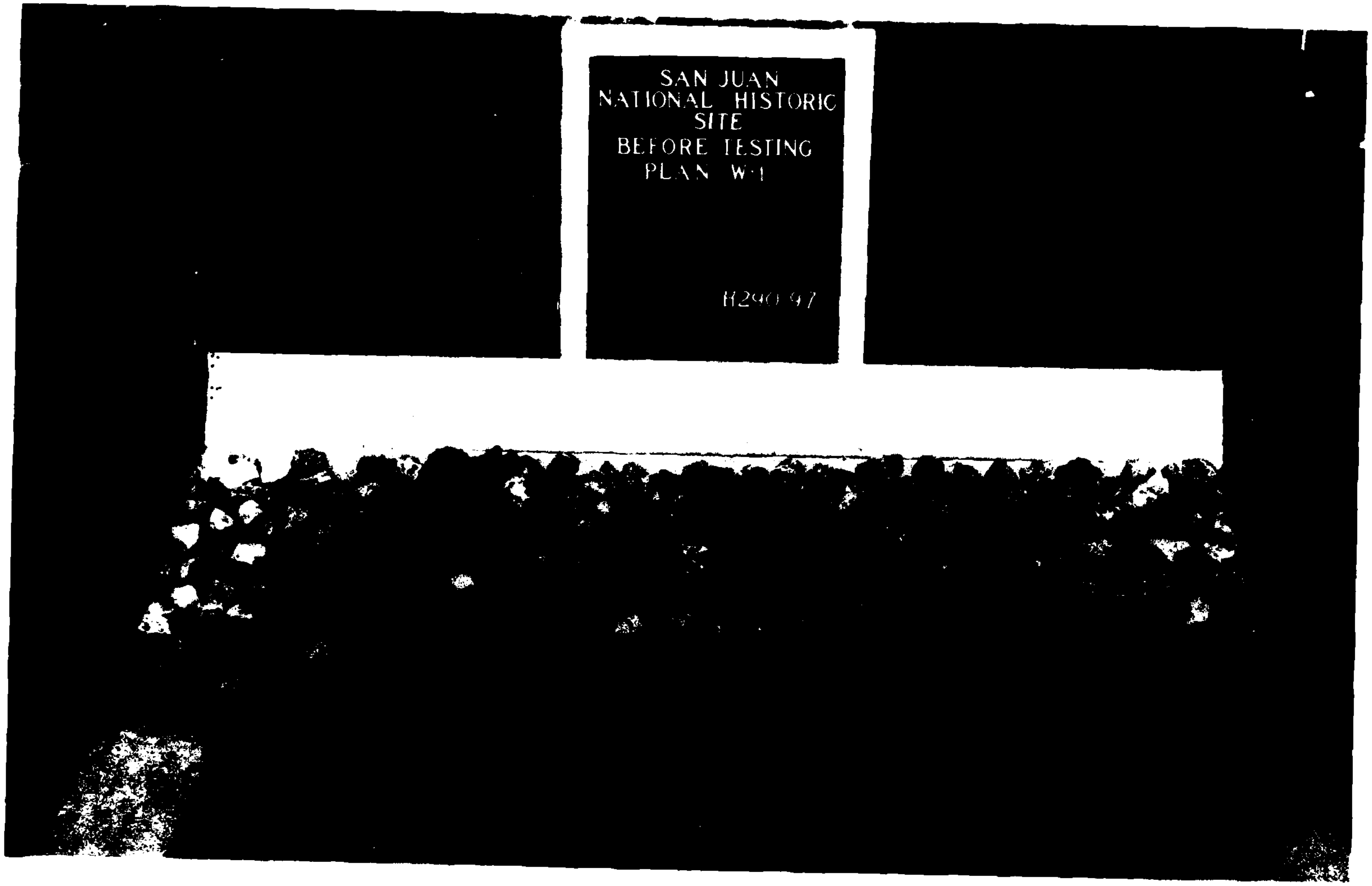

Photo 95. Sea-side view of Plan W-1 before testing, 1st test section

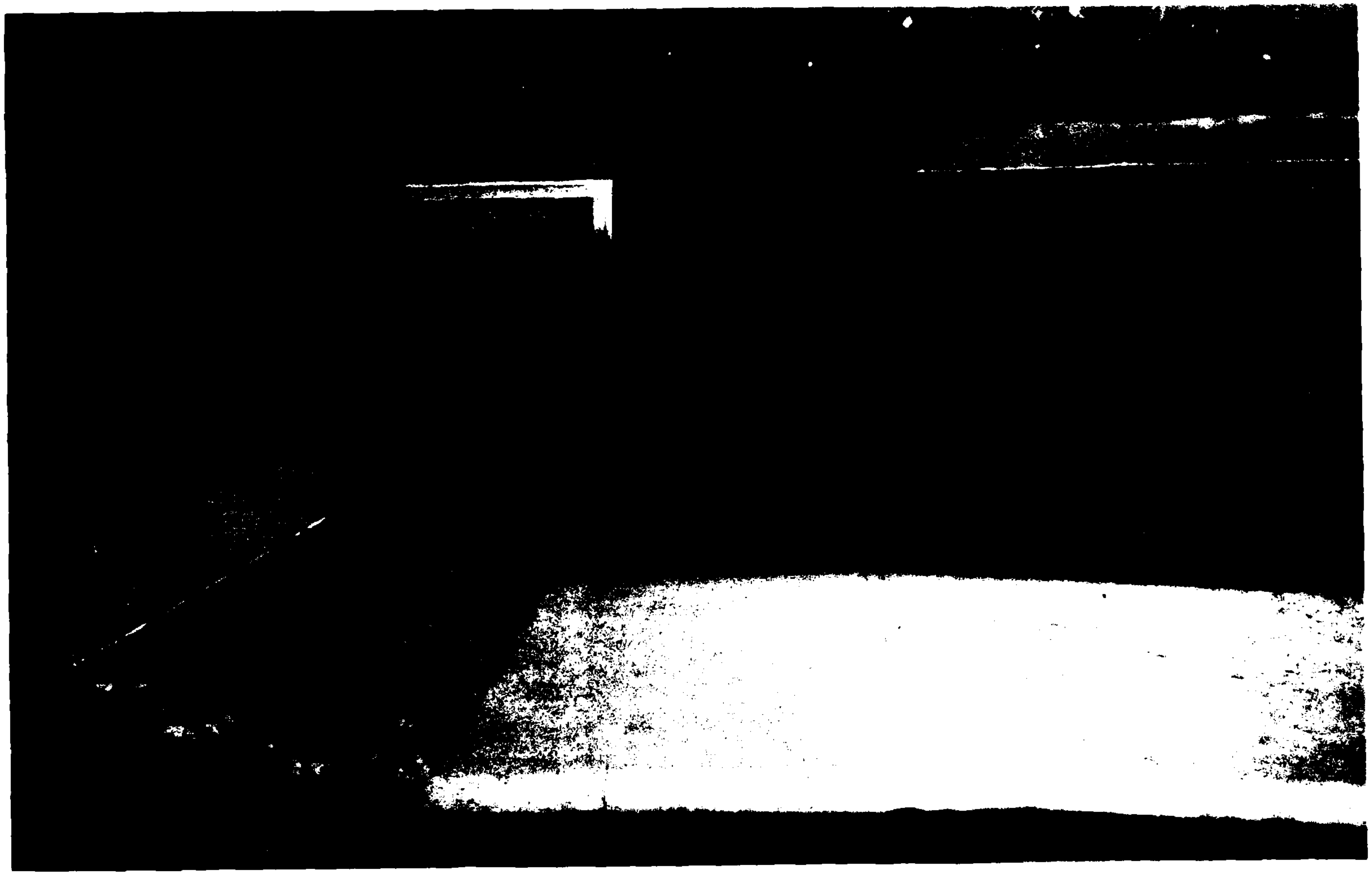

Photo 96. Side view of Plan W-1 after testing Hydrograph B (Plate 2 and Table 2), 1st test section

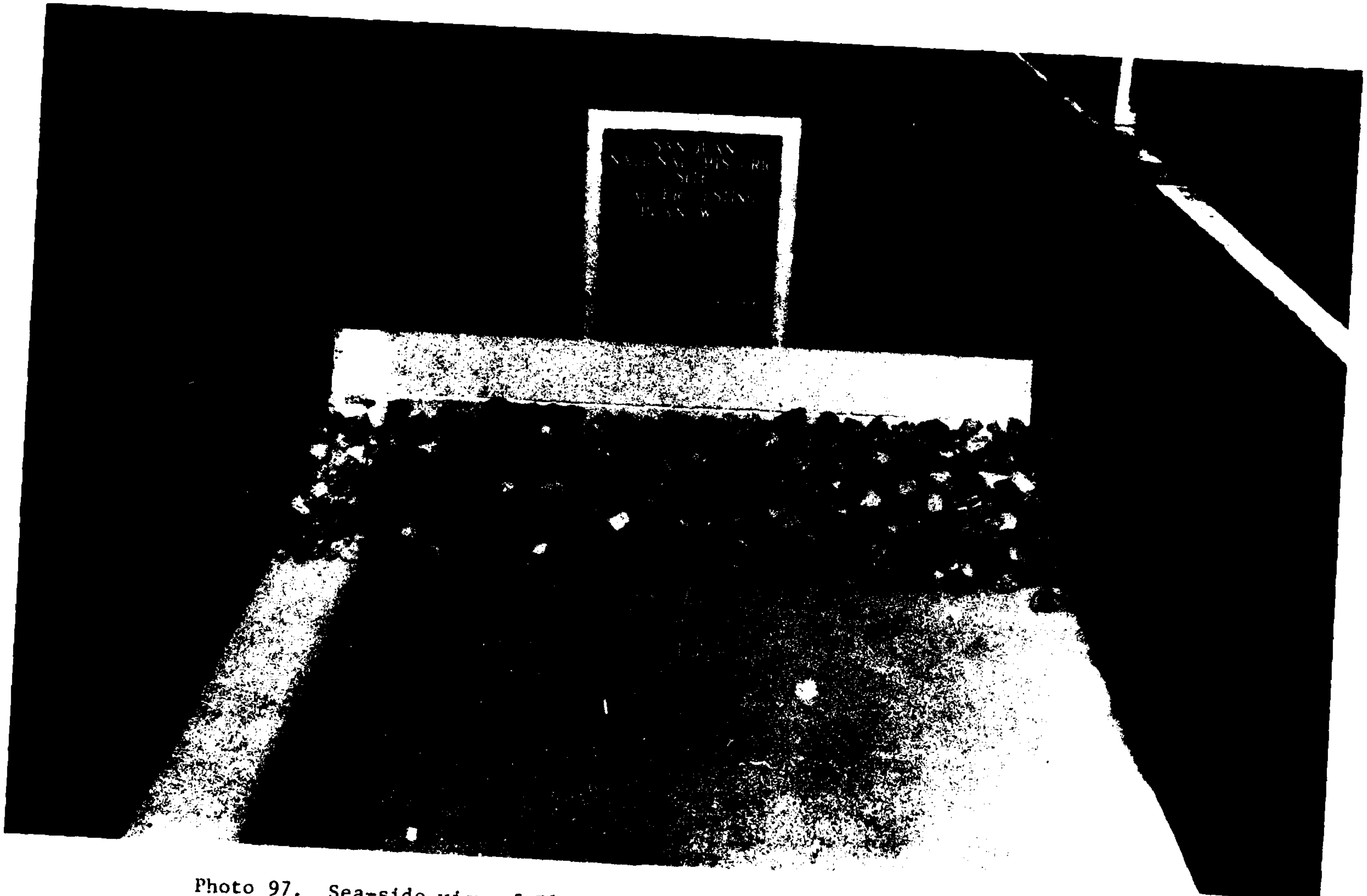

Photo 97. Sea-side view of Plan W-1 after testing Hydrograph B (Plate 2 and Table 2), 1st test section

Photo 98. Side view of Plan W-1 after testing Hydrograph B (Plate 2 and Table 2), 2nd test section

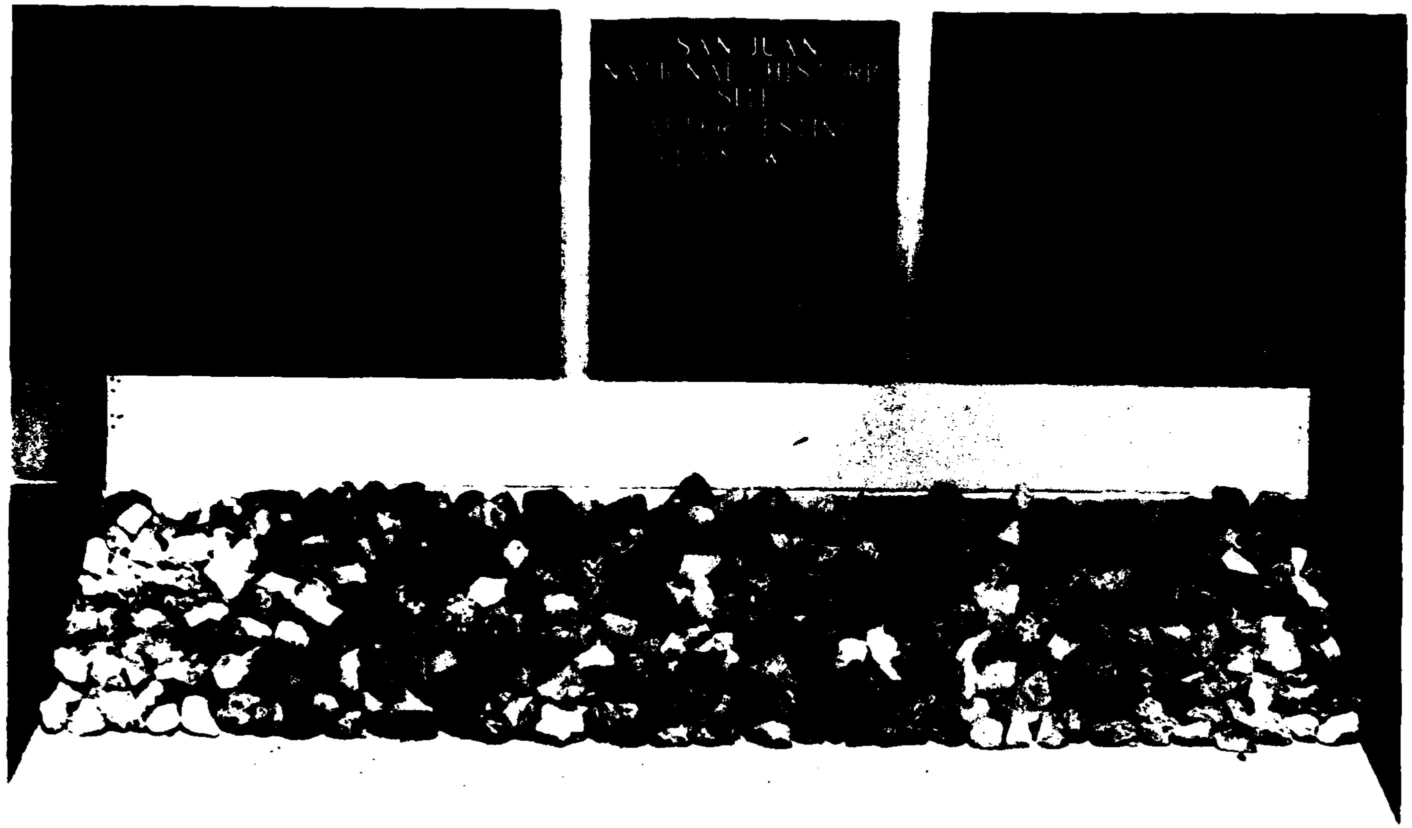

Photo 99. Sea-side view of Plan W-1 after testing Hydrograph B (Plate 2 and Table 2), 2nd test section

Photo 100. Side view of Plan W-2 before testing, 1st test section

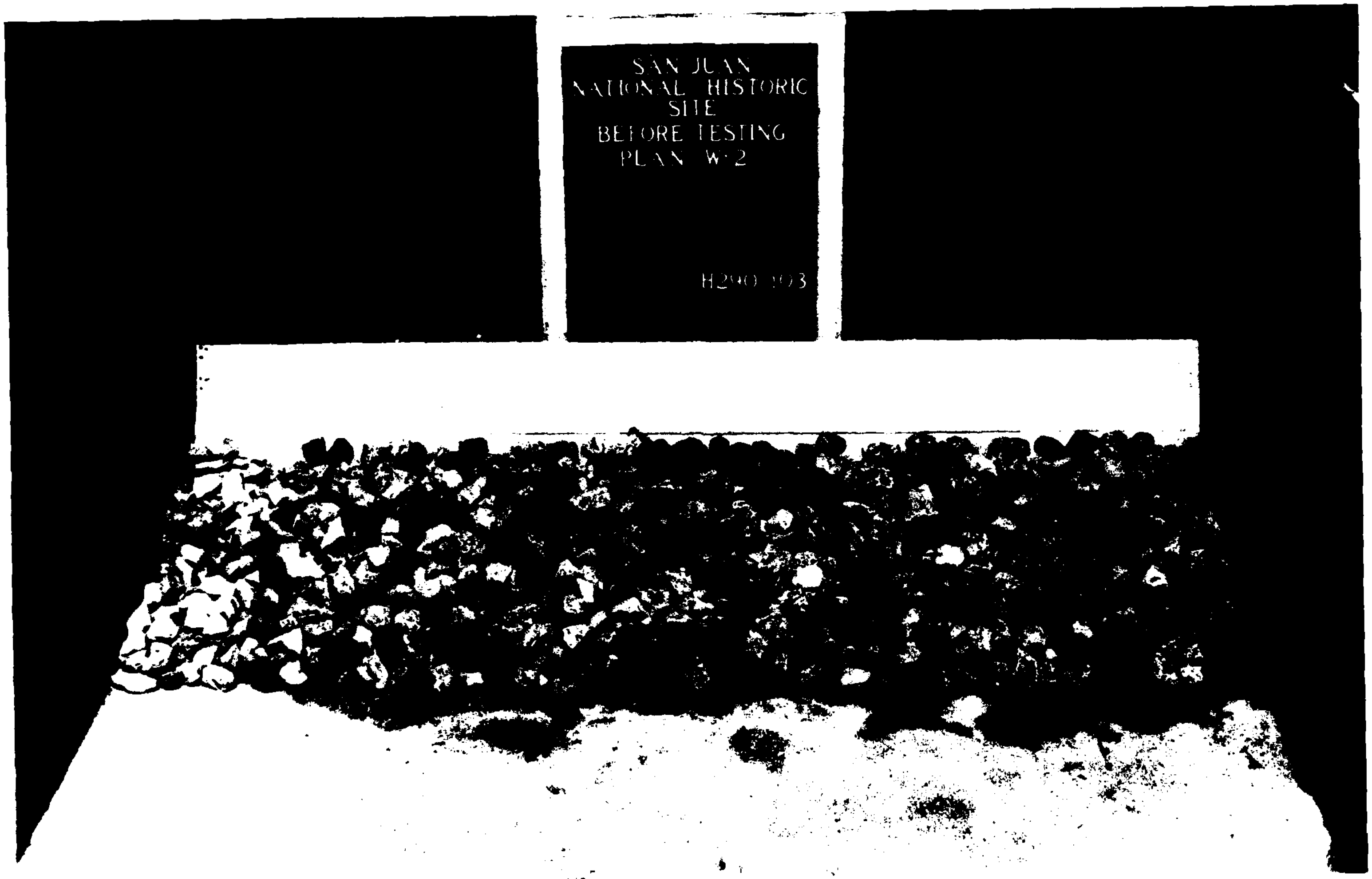

Photo 101. Sea-side view of Plan W-2 before testing, 1st test section

Photo 102. Side view of Plan W-2 after testing Hydrograph B (Plate 2 and Table 2), 1st test section

Photo 103. Sea-side view of Plan W-2 after testing Hydrograph B (Plate 2 and Table 2), 1st test section

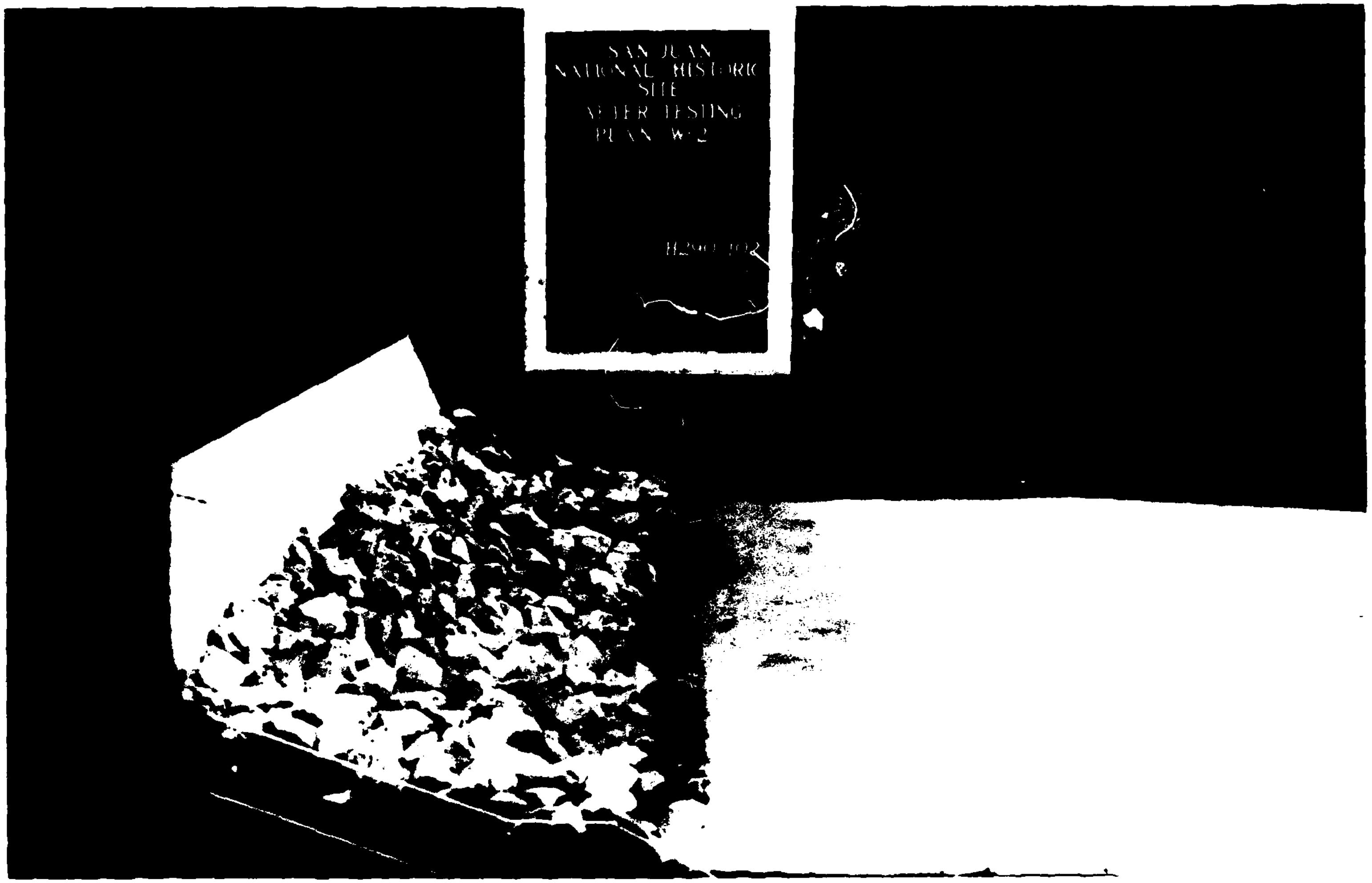

Photo 104. *Side view of Plan W-2 after testing Hydrograph B (Plate 2 and Table 2), 2nd test section*

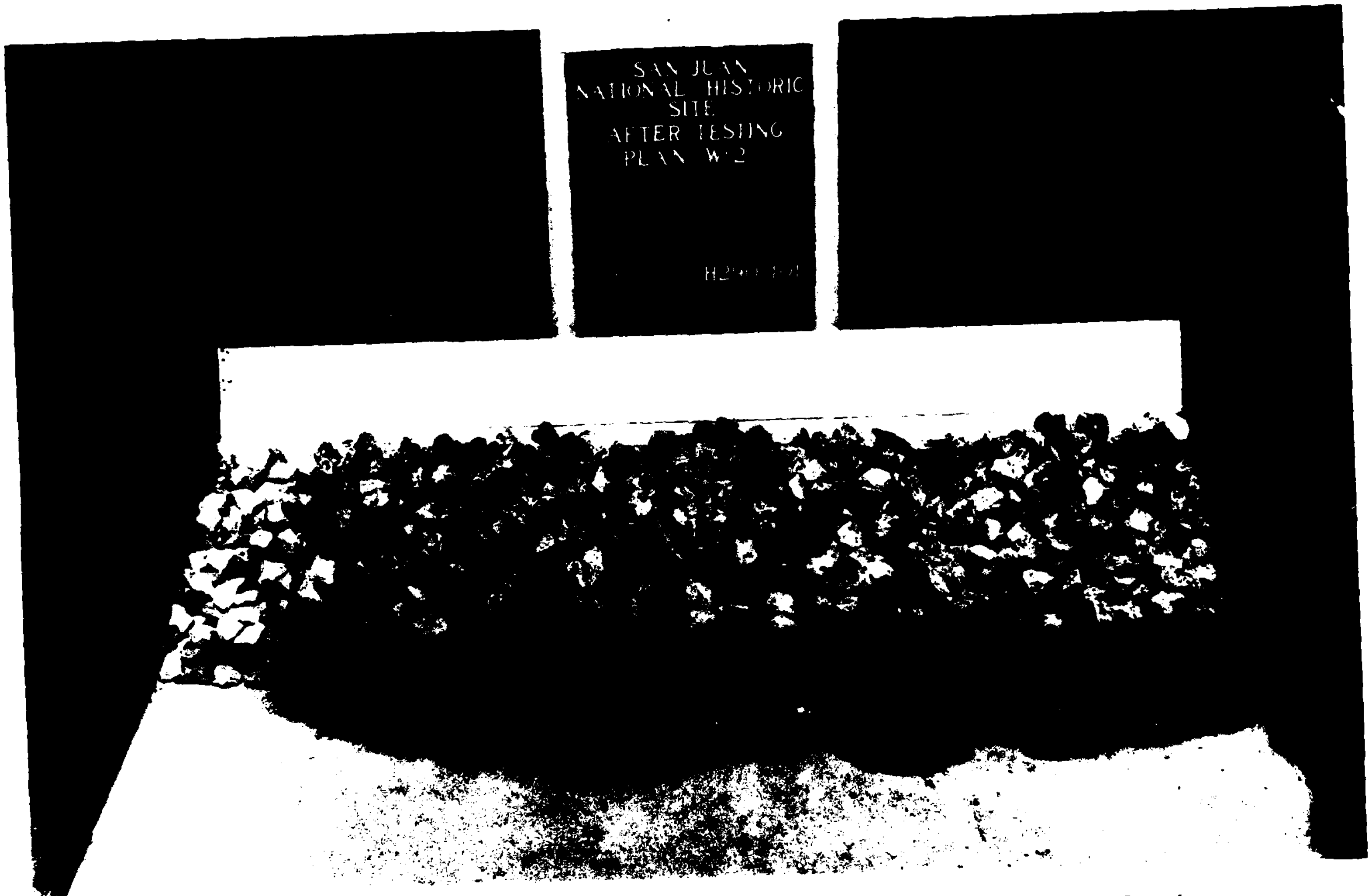

Photo 105. Sea-side view of Plan W-2 after testing Hydrograph B (Plate 2 and Table 2), 2nd test section

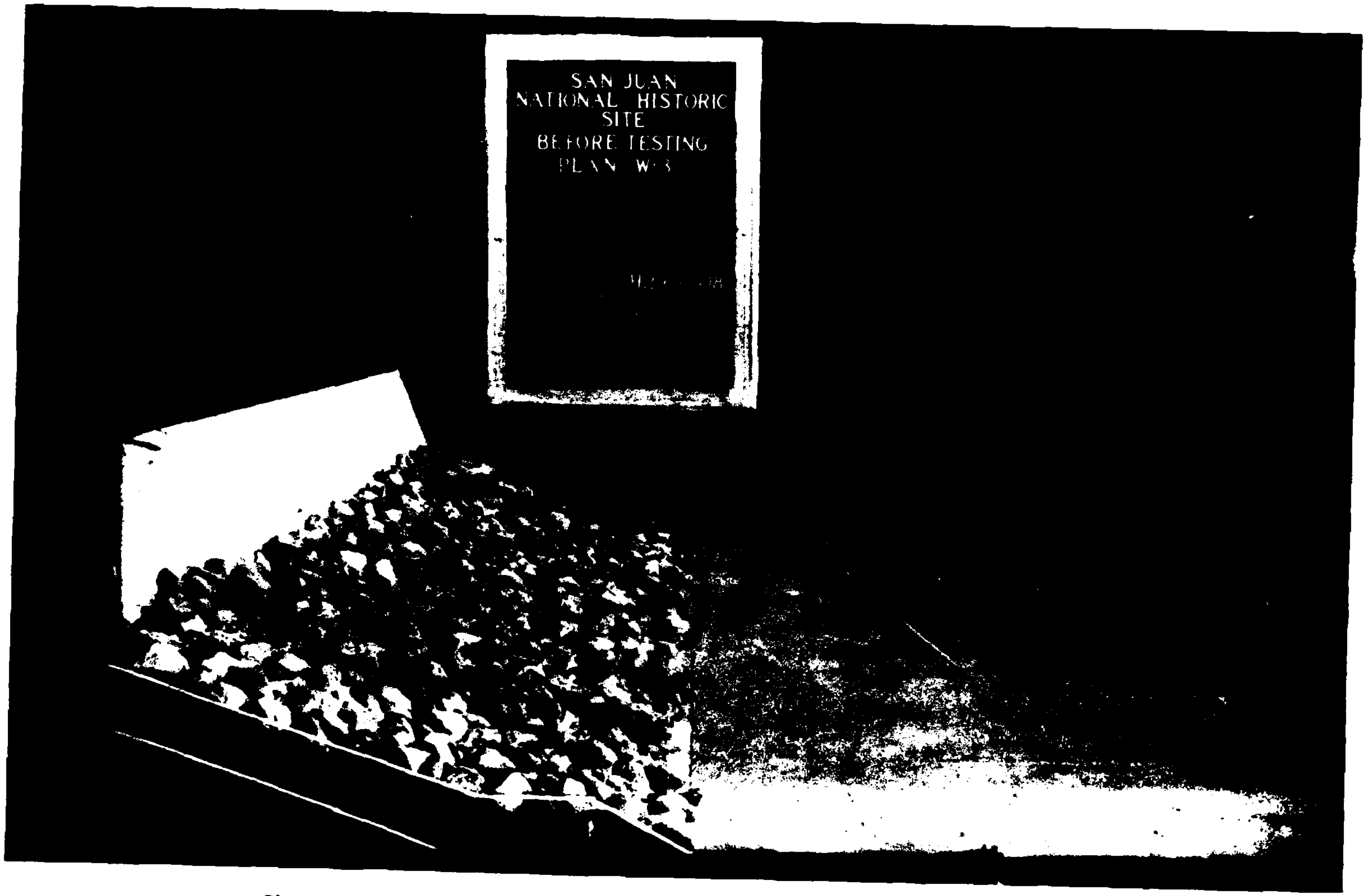

Photo 106. *Side view of Plan W-3 before testing, 1st test section*

Photo 107. Sea-side view of Plan W-3 before testing, 1st test section

Photo 108. Side view of Plan W-3 after testing Hydrograph B (Plate 2 and Table 2), 1st test section

Photo 109. Sea-side view of Plan W-3 after testing Hydrograph B (Plate 2 and Table 2), 1st test section

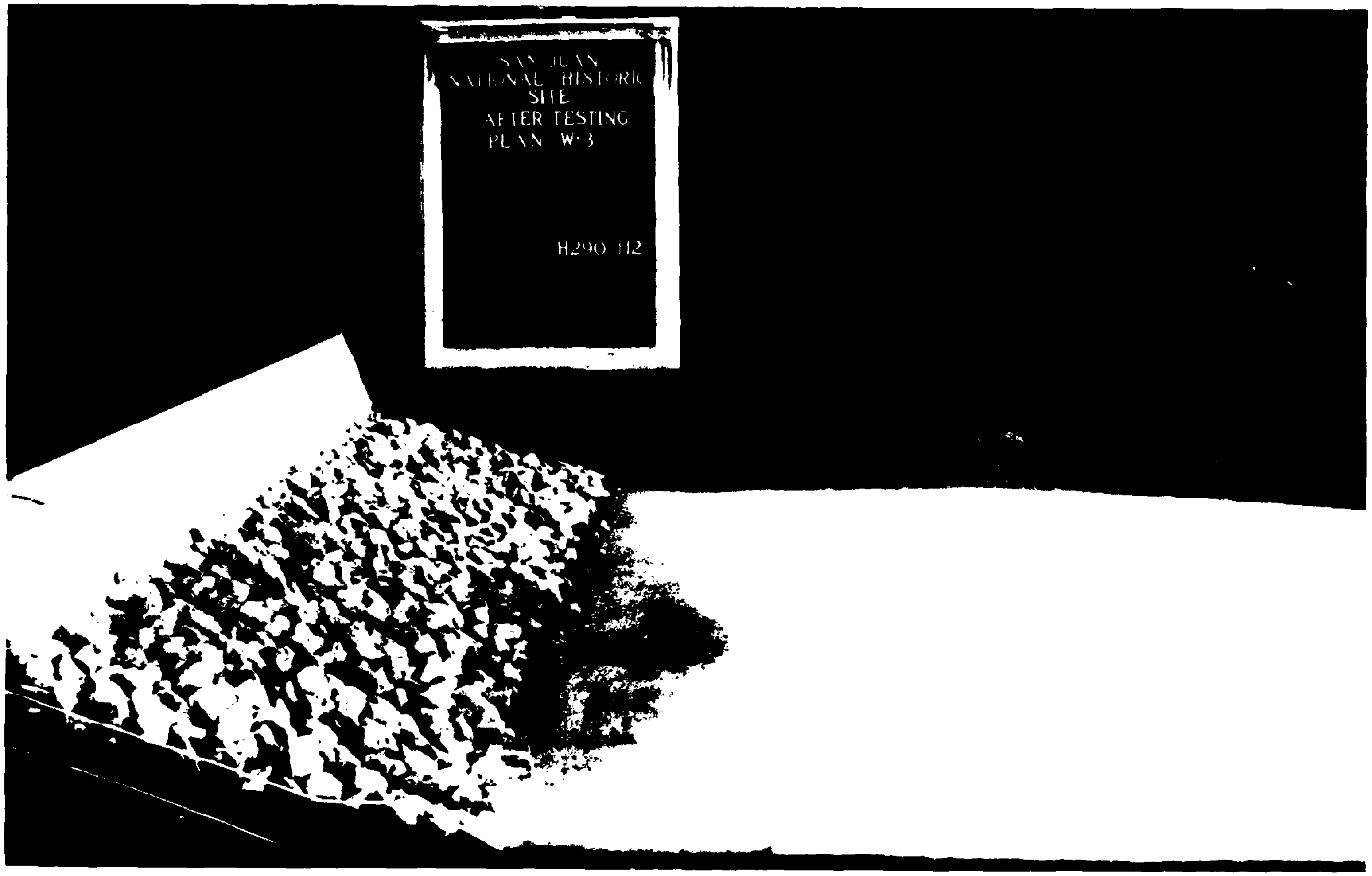

Photo 110. Side view of Plan W-3 after testing Hydrograph B (Plate 2 and Table 2), 2nd test section

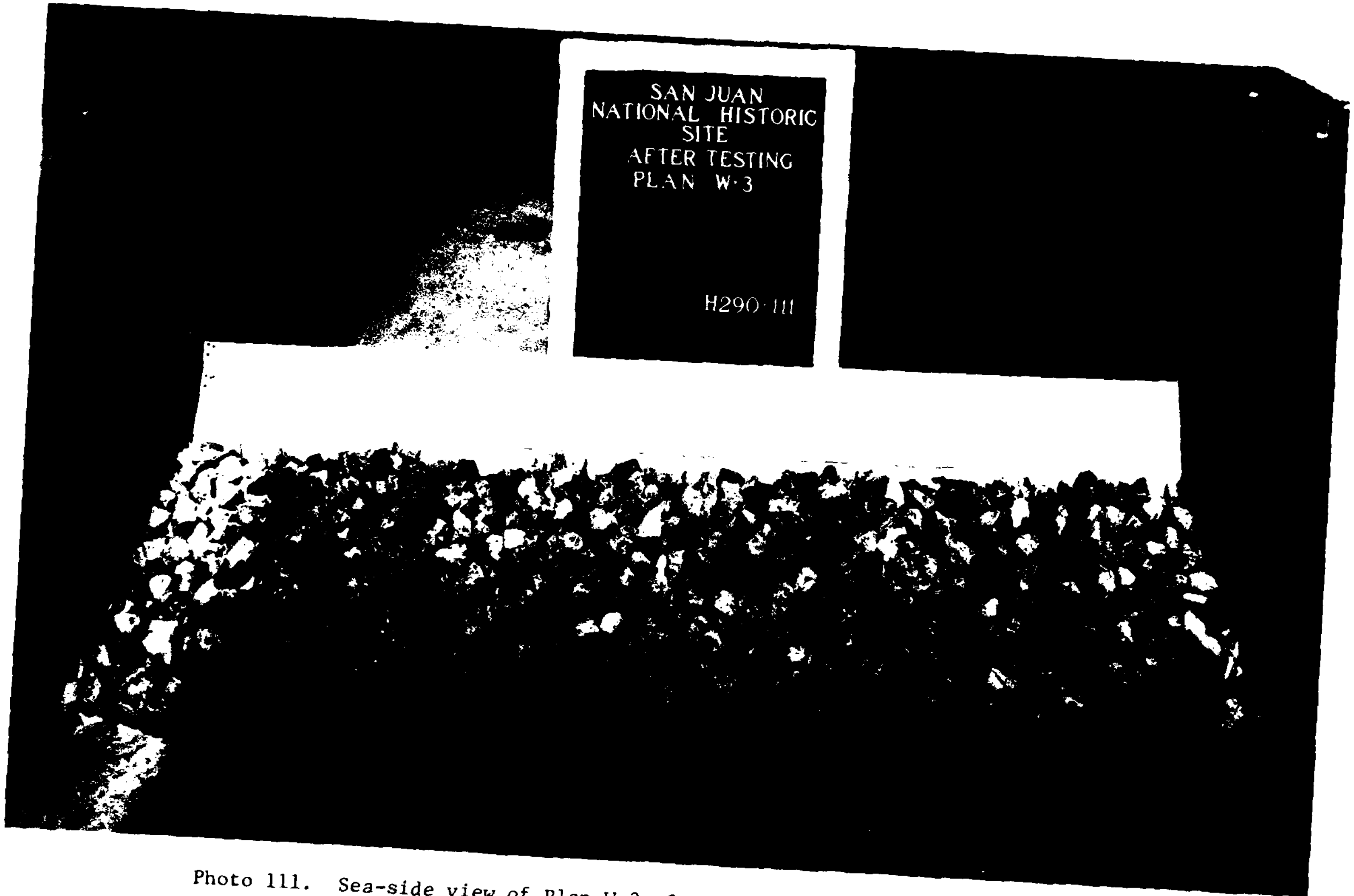

Photo 111. Sea-side view of Plan W-3 after testing Hydrograph B (Plate 2 and Table 2), 2nd test section

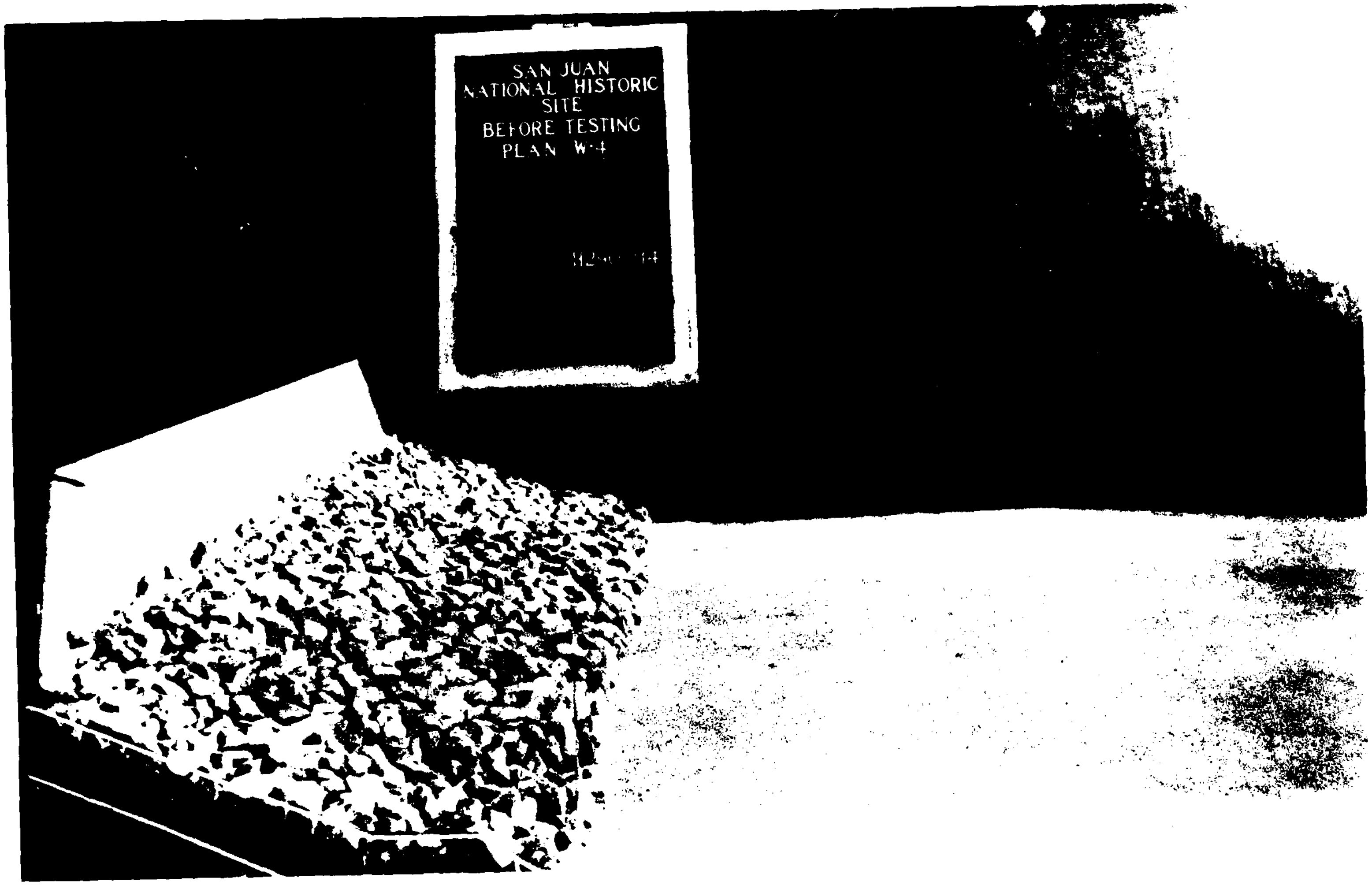

Photo 112. Side view of Plan W-4 before testing, 1st test section

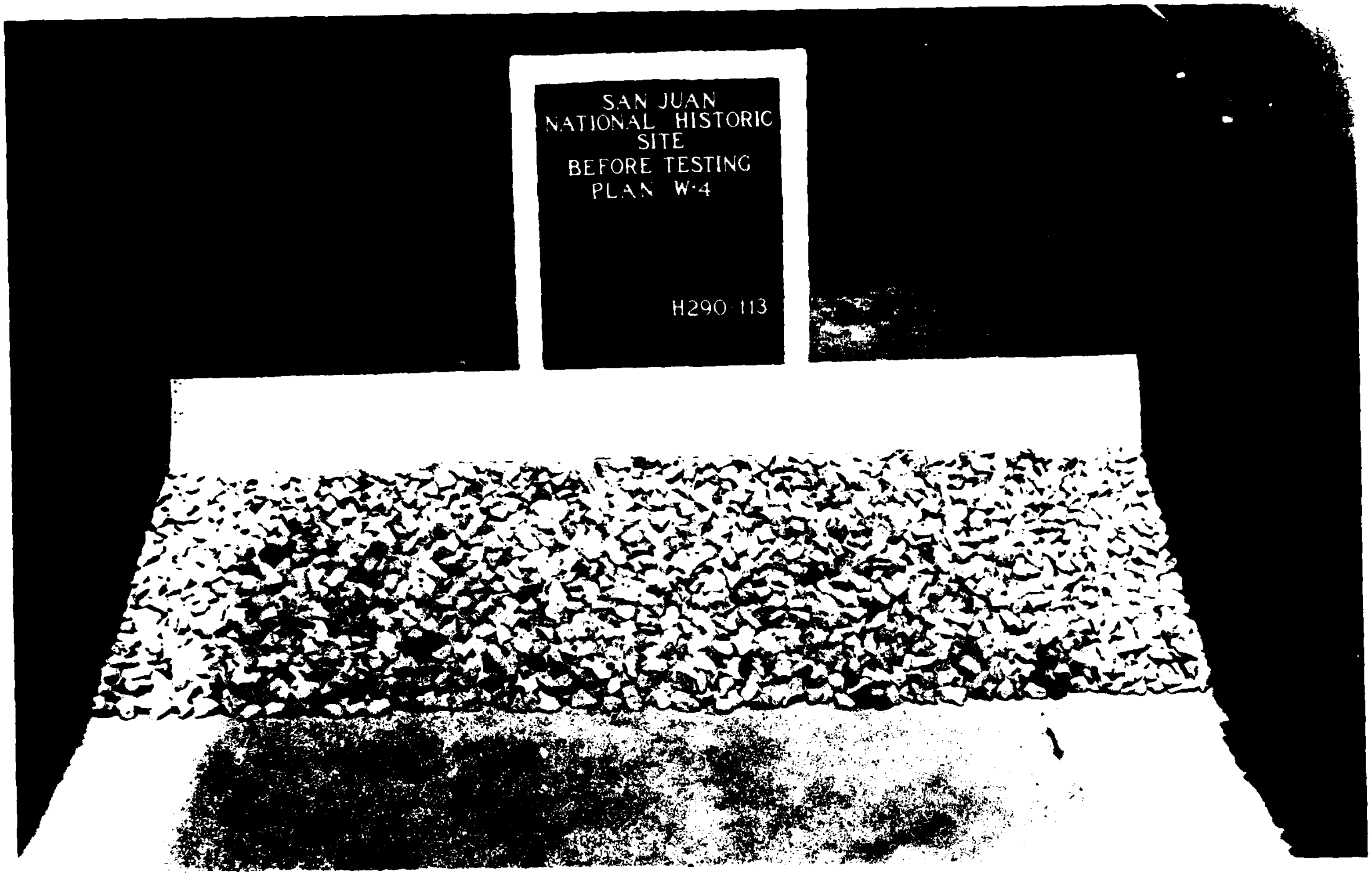

Photo 113. Sea-side view of Plan W-4 before testing, 1st test section

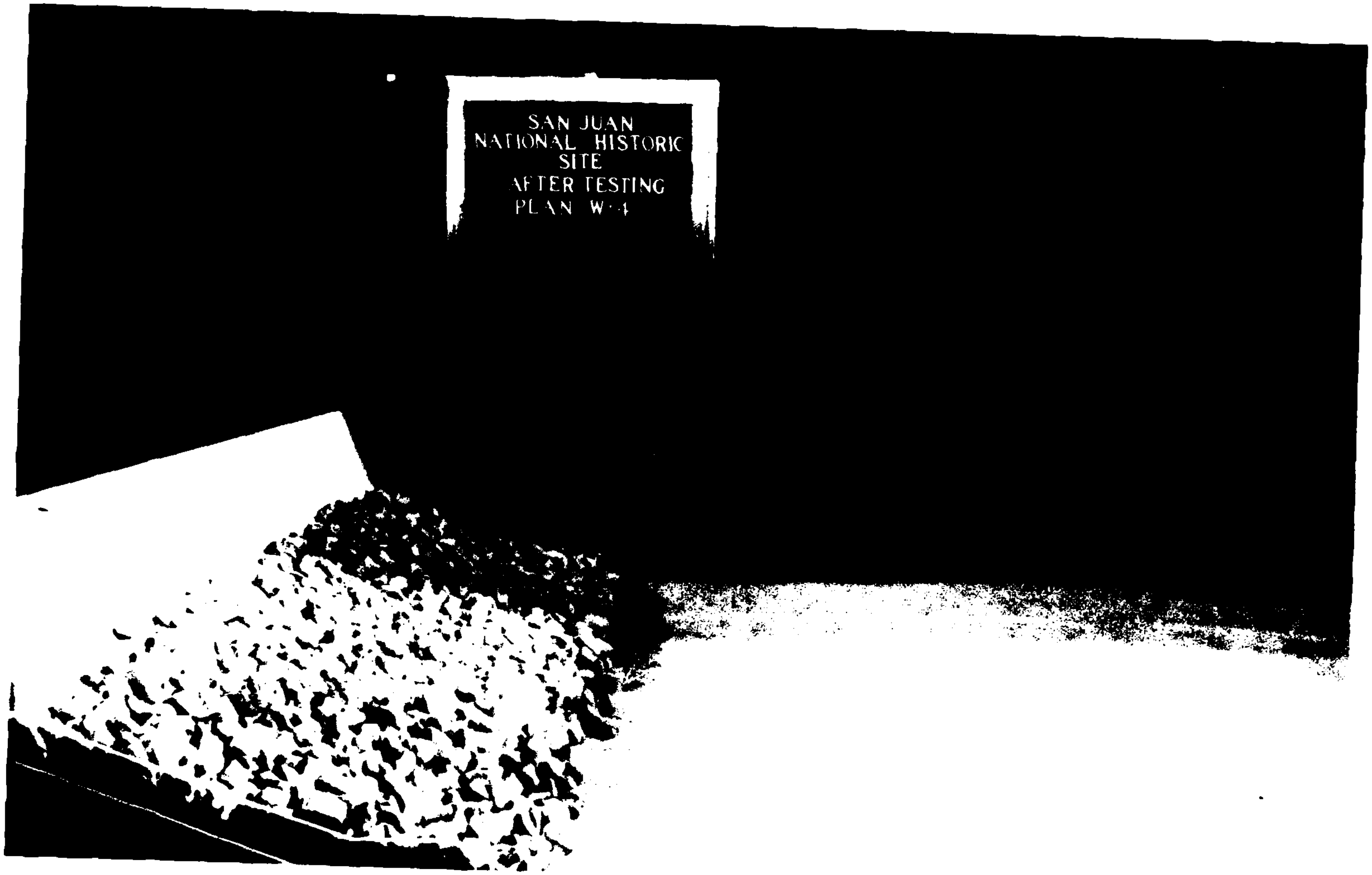

Photo 114. Side view of Plan W-4 after testing Hydrograph B (Plate 2 and Table 2), 1st test section

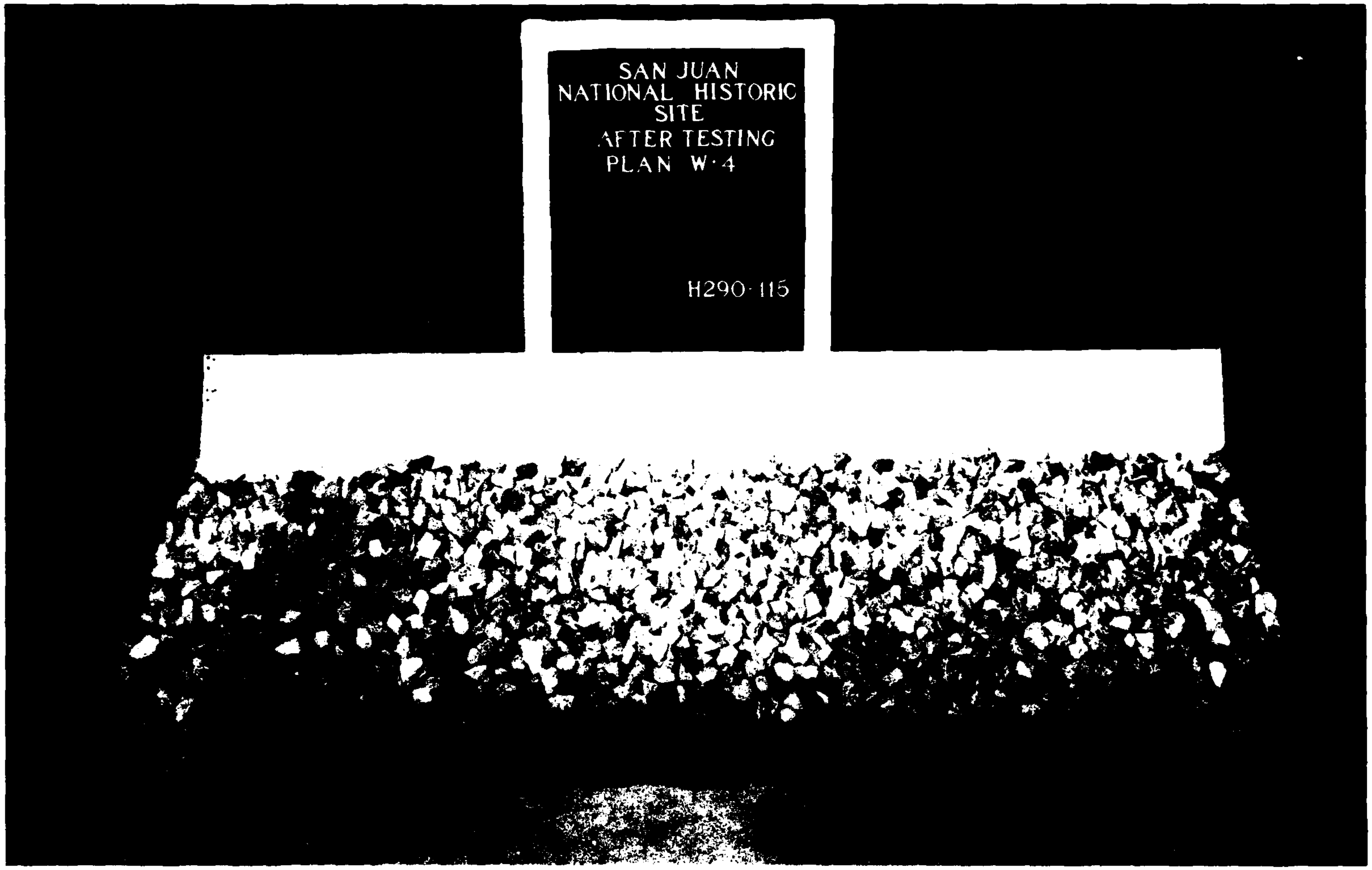

Photo 115. Sea-side view of Plan W-4 after testing Hydrograph B (Plate 2 and Table 2), 1st test section

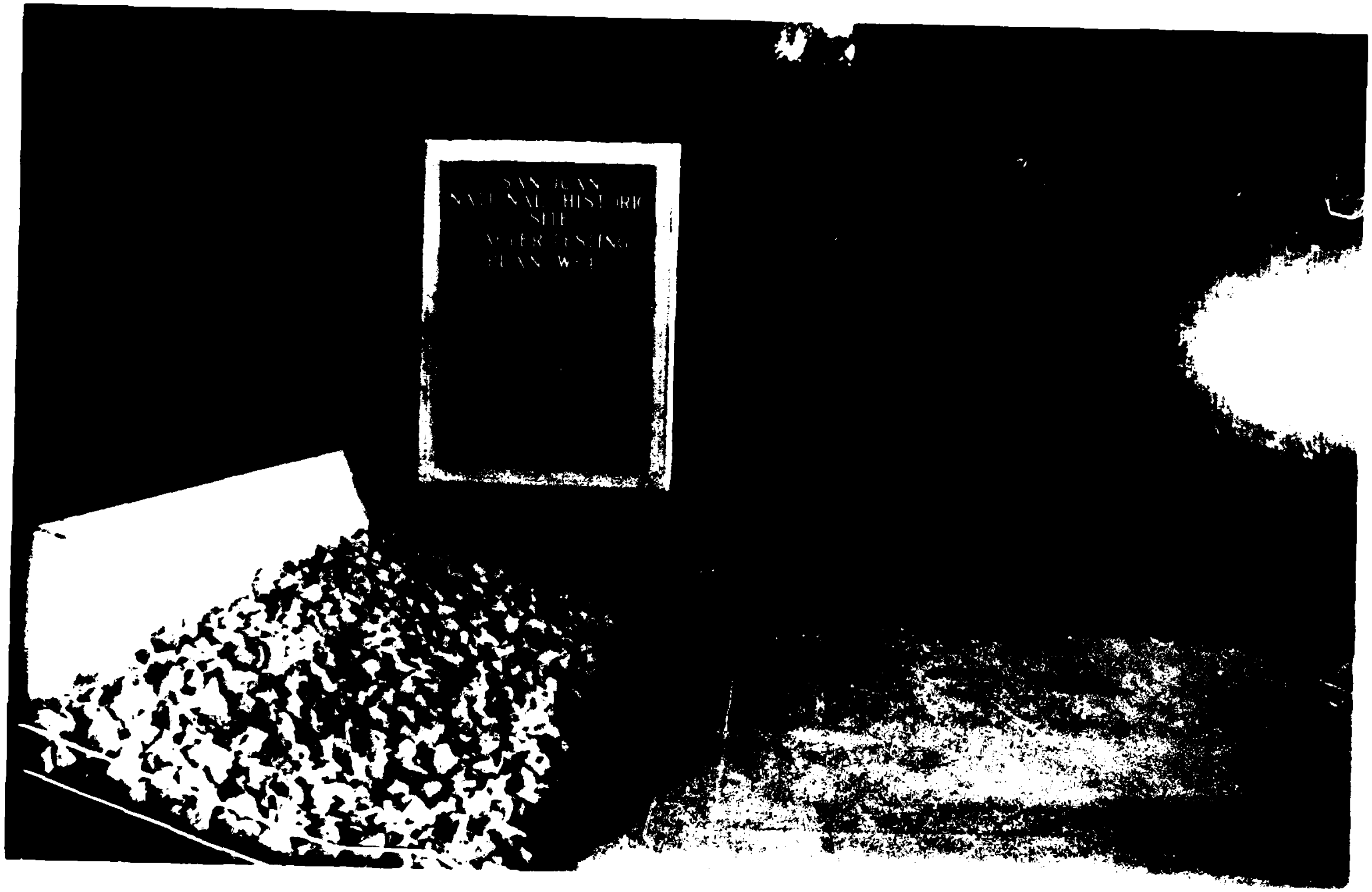

Photo 116. Side view of Plan W-4 after testing Hydrograph B (Plate 2 and Table 2), 2nd test section

Photo 117. Sea-side view of Plan W-4 after testing Hydrograph B (Plate 2 and Table 2), 2nd test section

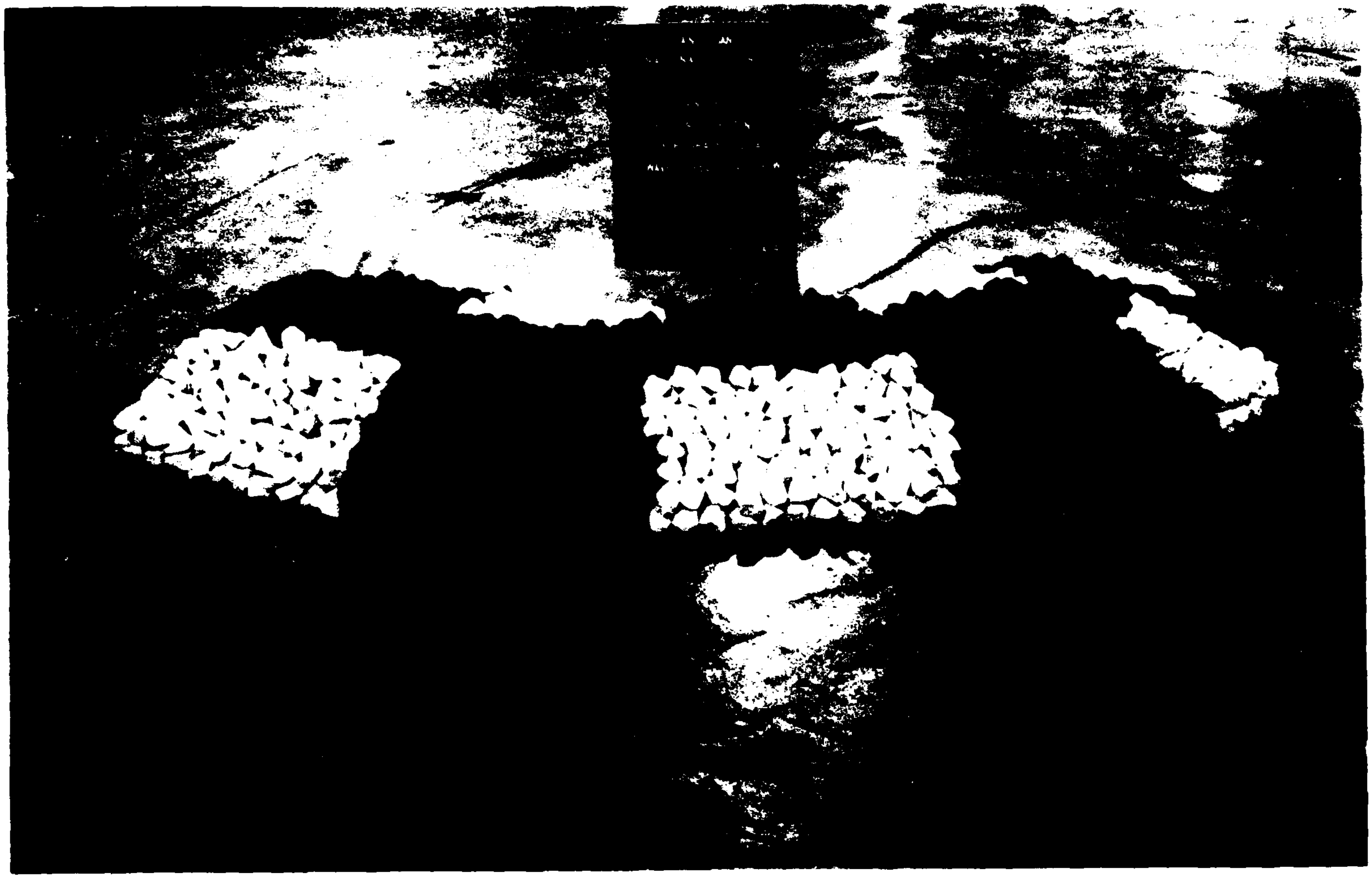

Photo 118. Sea-side view of Plan 3D-1 before testing N30°W wave direction, 1st test section

Photo 119. End view of Plan 3D-1 before testing N30°W wave direction, 1st test section

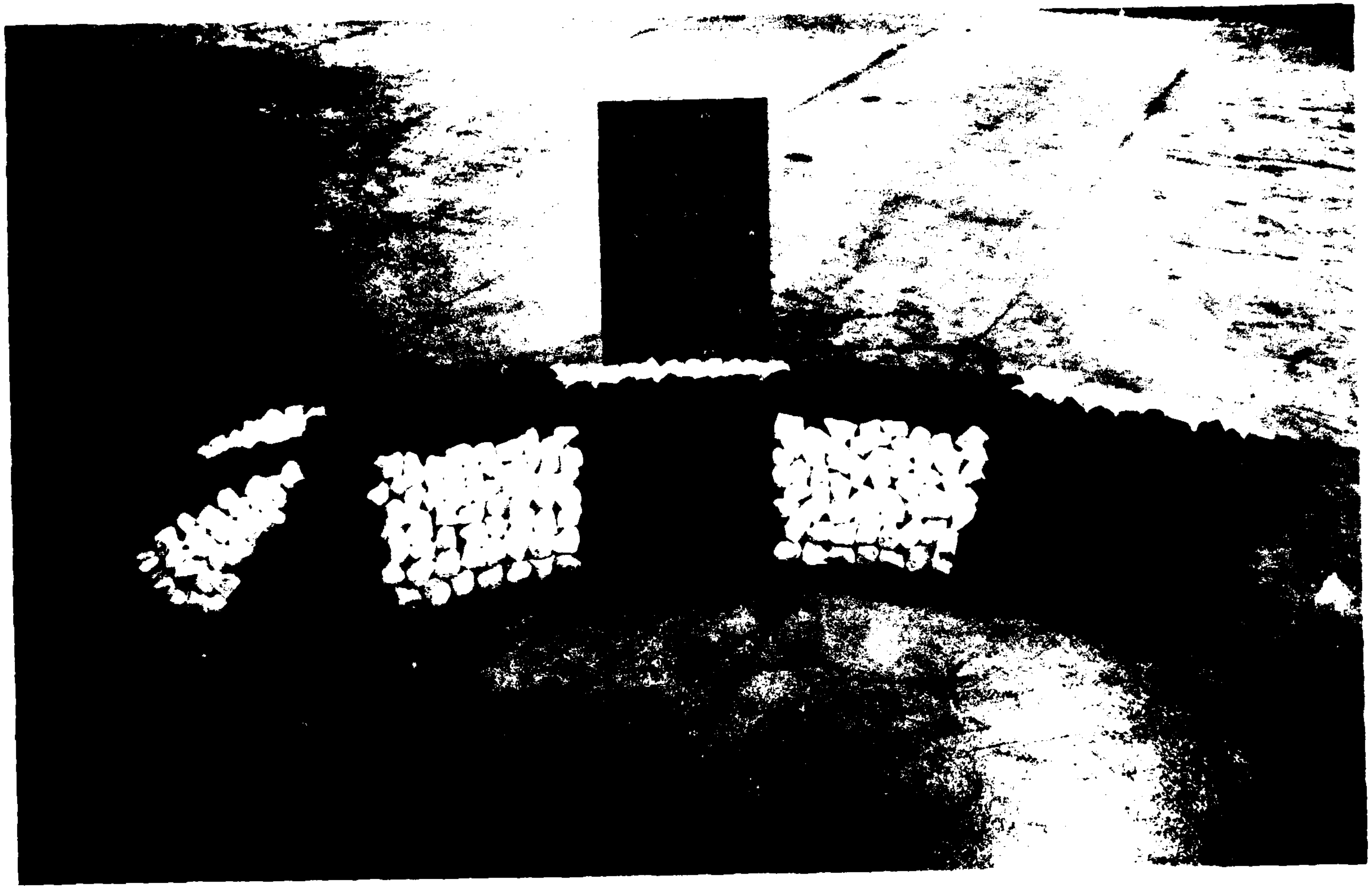

Photo 120. Beach-side view of Plan 3D-1 before testing N30°W wave direction, 1st test section

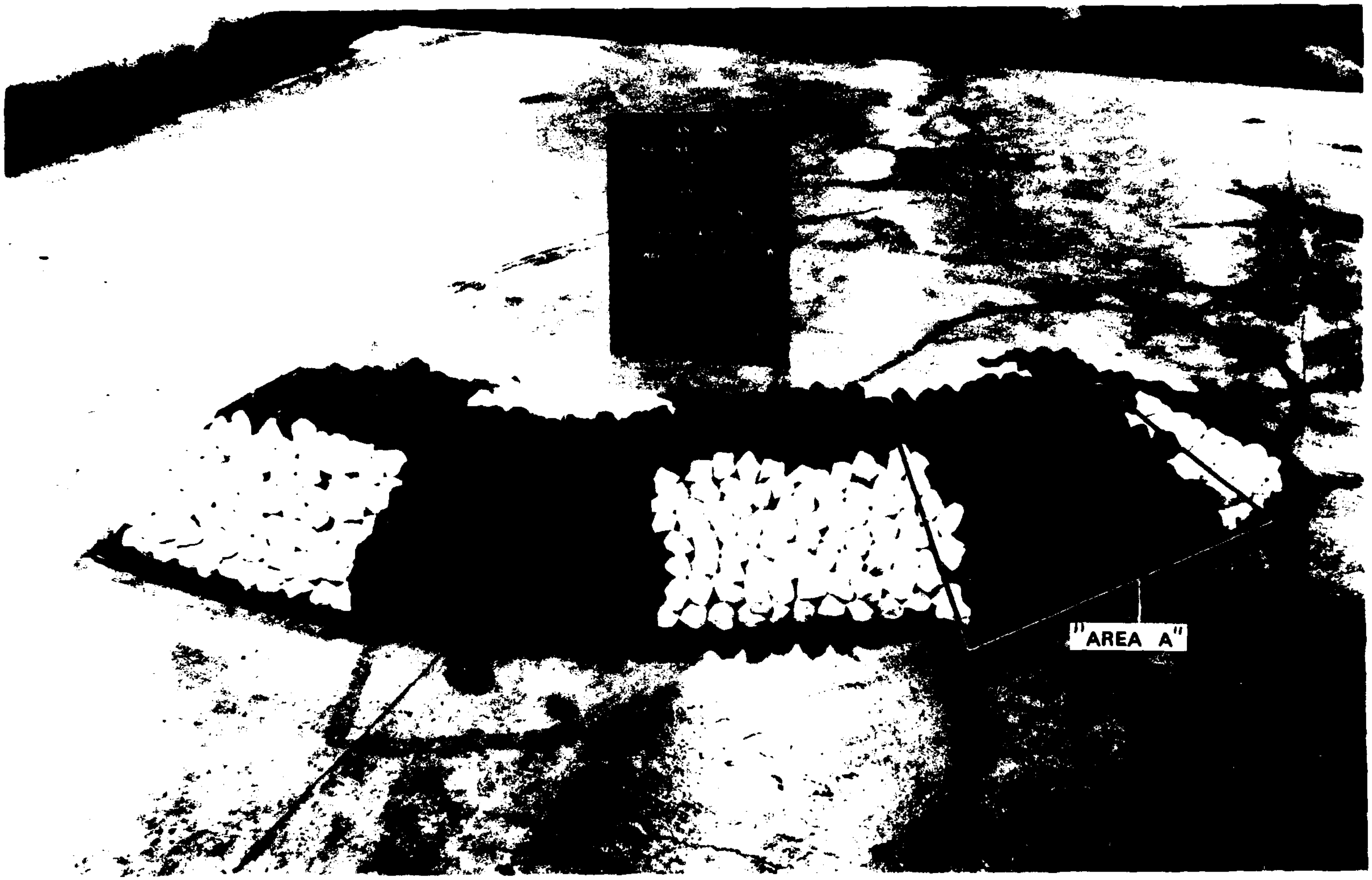

Photo 121. Sea-side view of Plan 3D-1 after testing Hydrograph 3D-A (Plate 3 and Table 3) for N30°W wave direction, 1st test section

Photo 122. End view of Plan 3D-1 after testing Hydrograph 3D-A (Plate 3 and Table 3) for N30°W wave direction, 1st test section

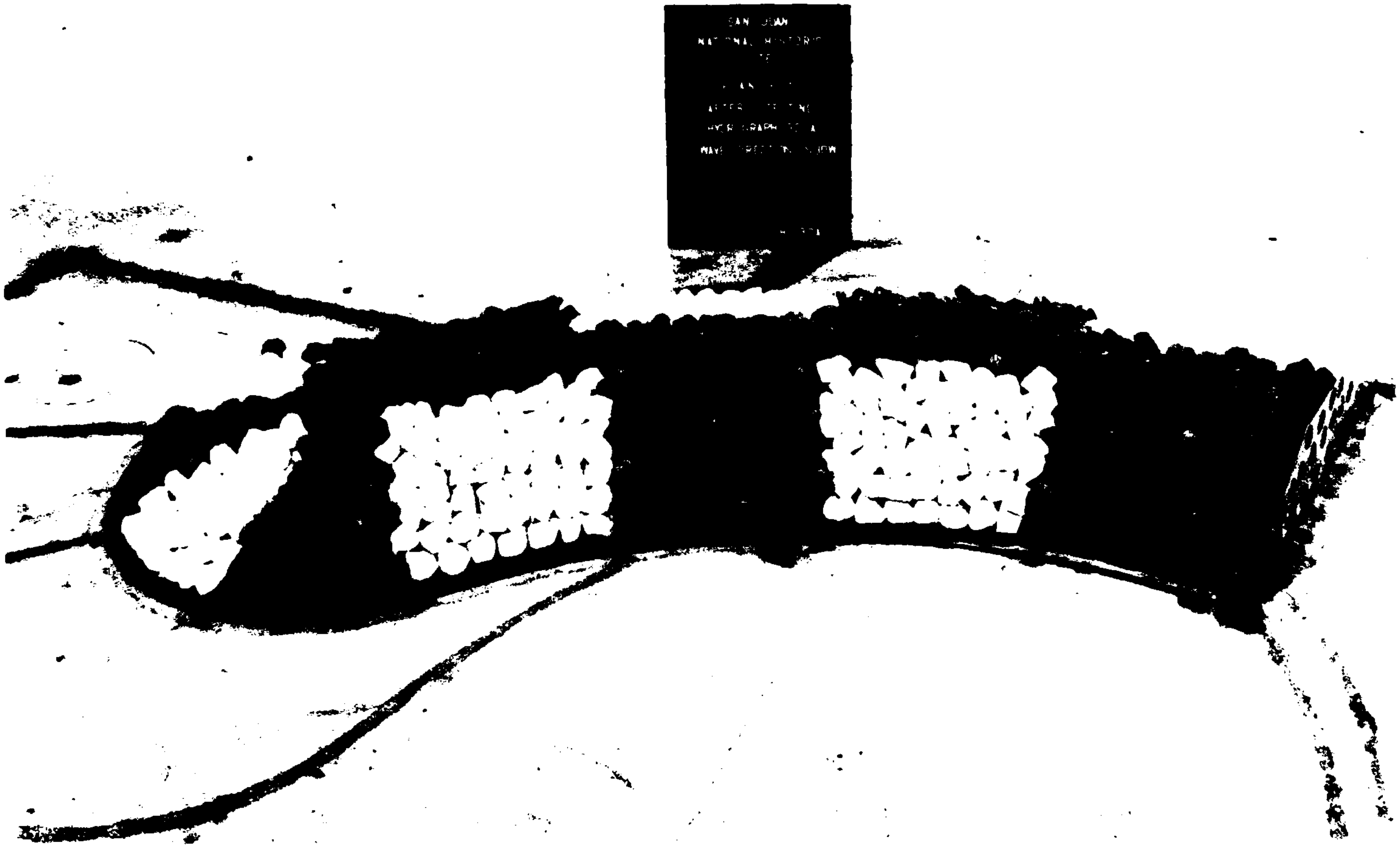

Photo 123. Beach-side view of Plan 3D-1 after testing Hydrograph 3D-A (Plate 3 and Table 3) for N30°W wave direction, 1st test section

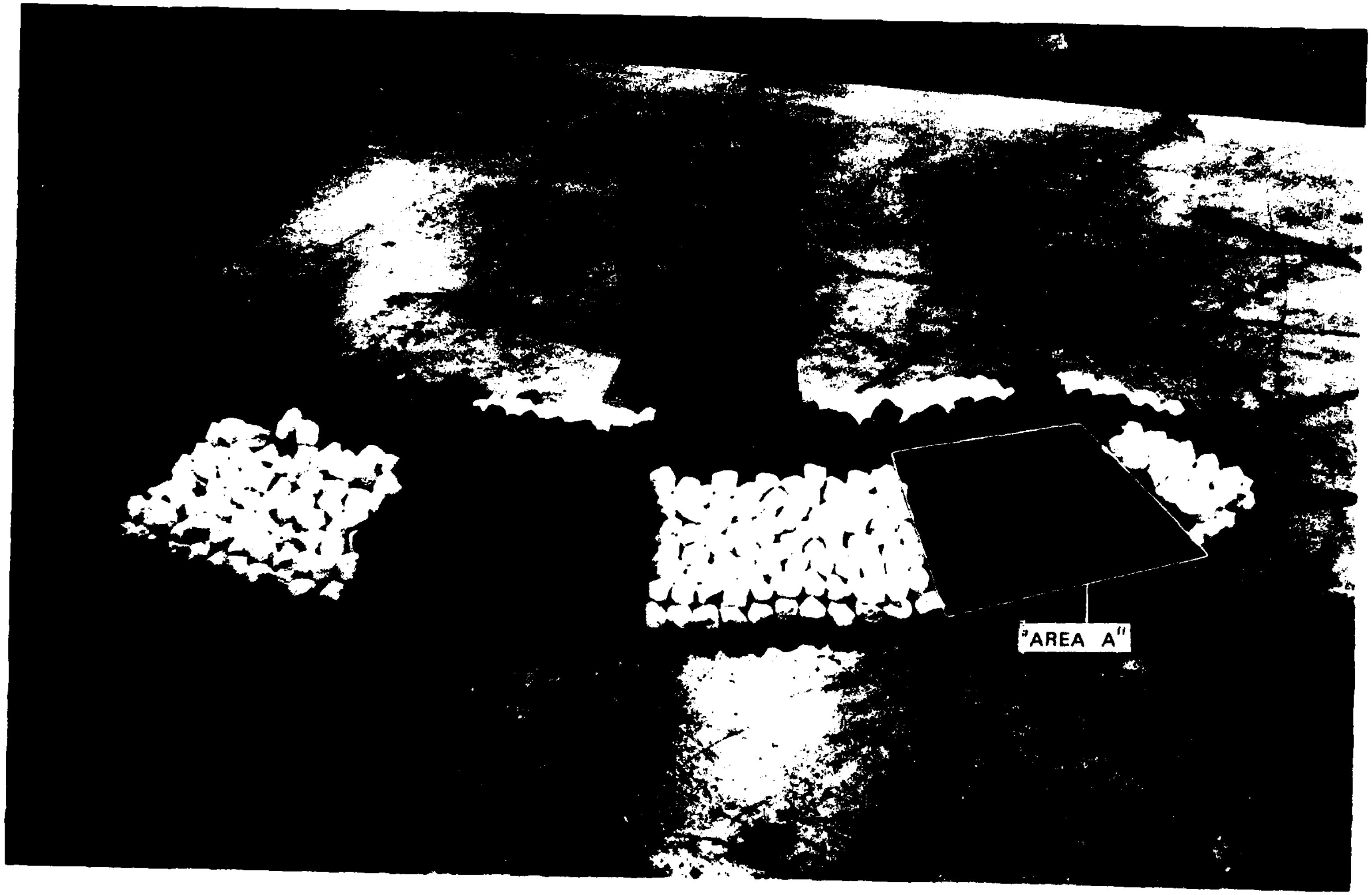

Photo 124. Sea-side view of Plan 3D-1 after testing Hydrograph 3D-A (Plate 3 and Table 3) for N30°W wave direction, 2nd test section

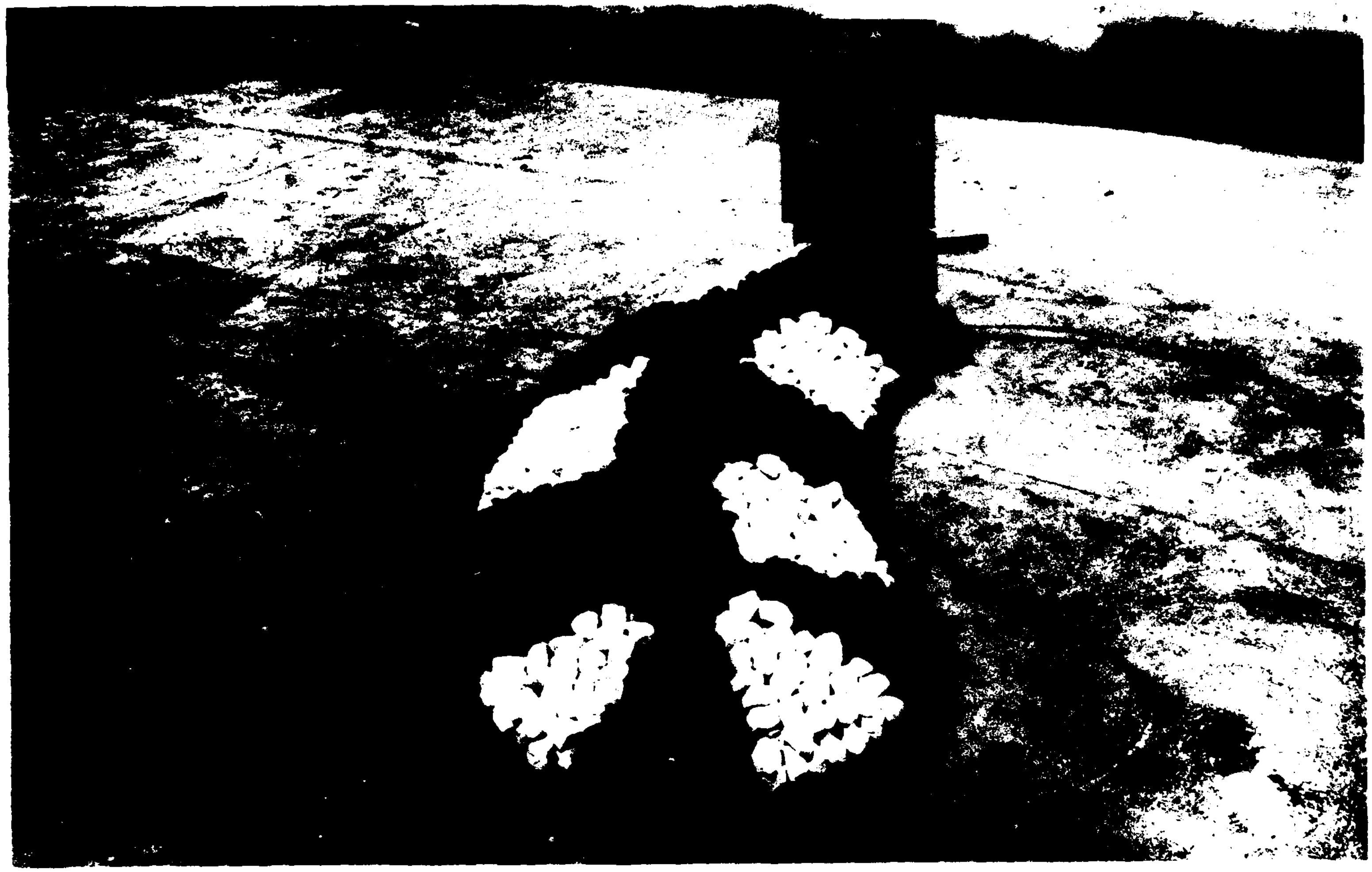

Photo 125. End view of Plan 3D-1 after testing Hydrograph 3D-A (Plate 3 and Table 3) for N30°W wave direction, 2nd test section

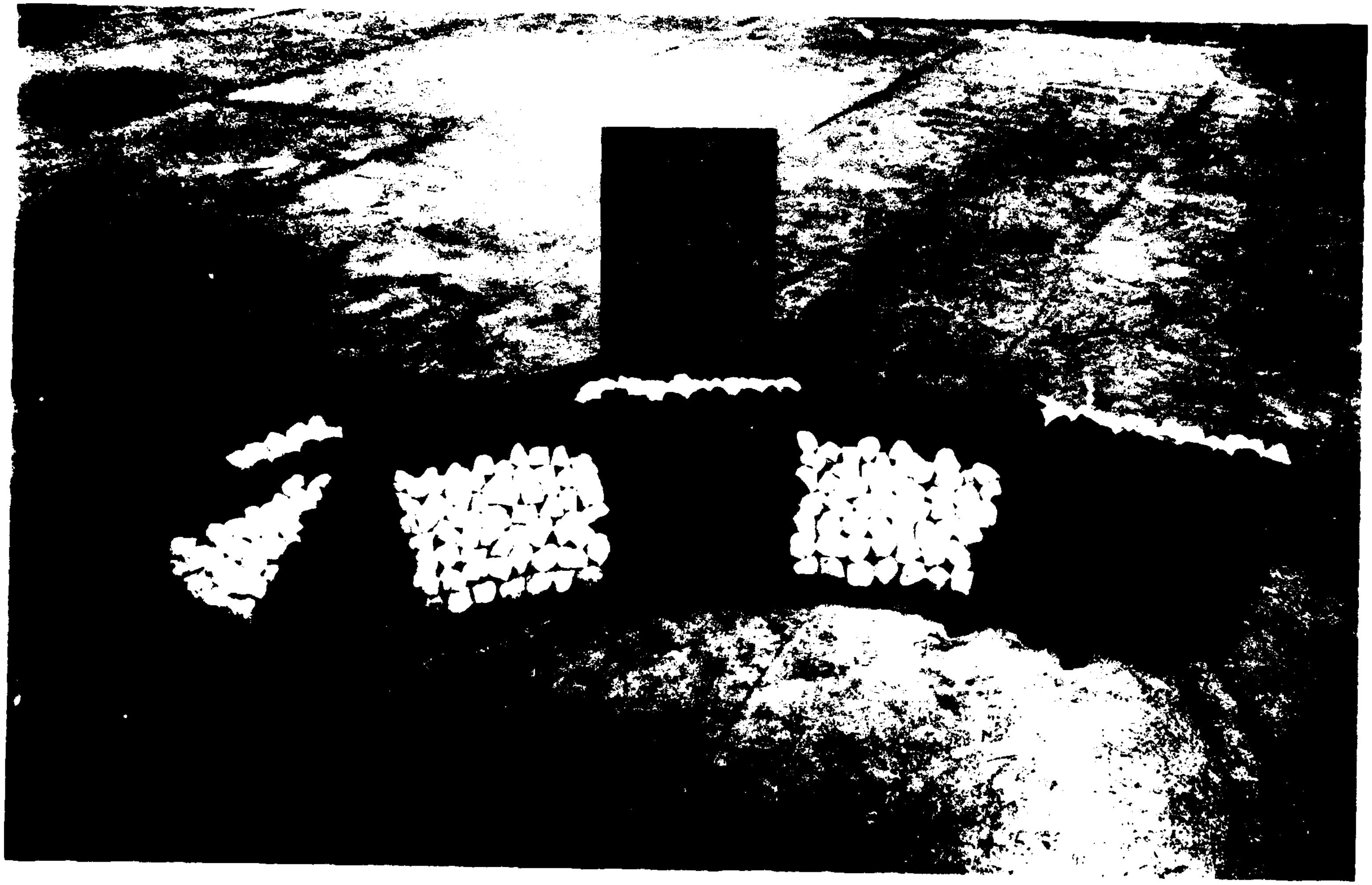

Photo 126. Beach-side view of Plan 3D-1 after testing Hydrograph 3D-A (Plate 3 and Table 3) for N30°W wave direction, 2nd test section

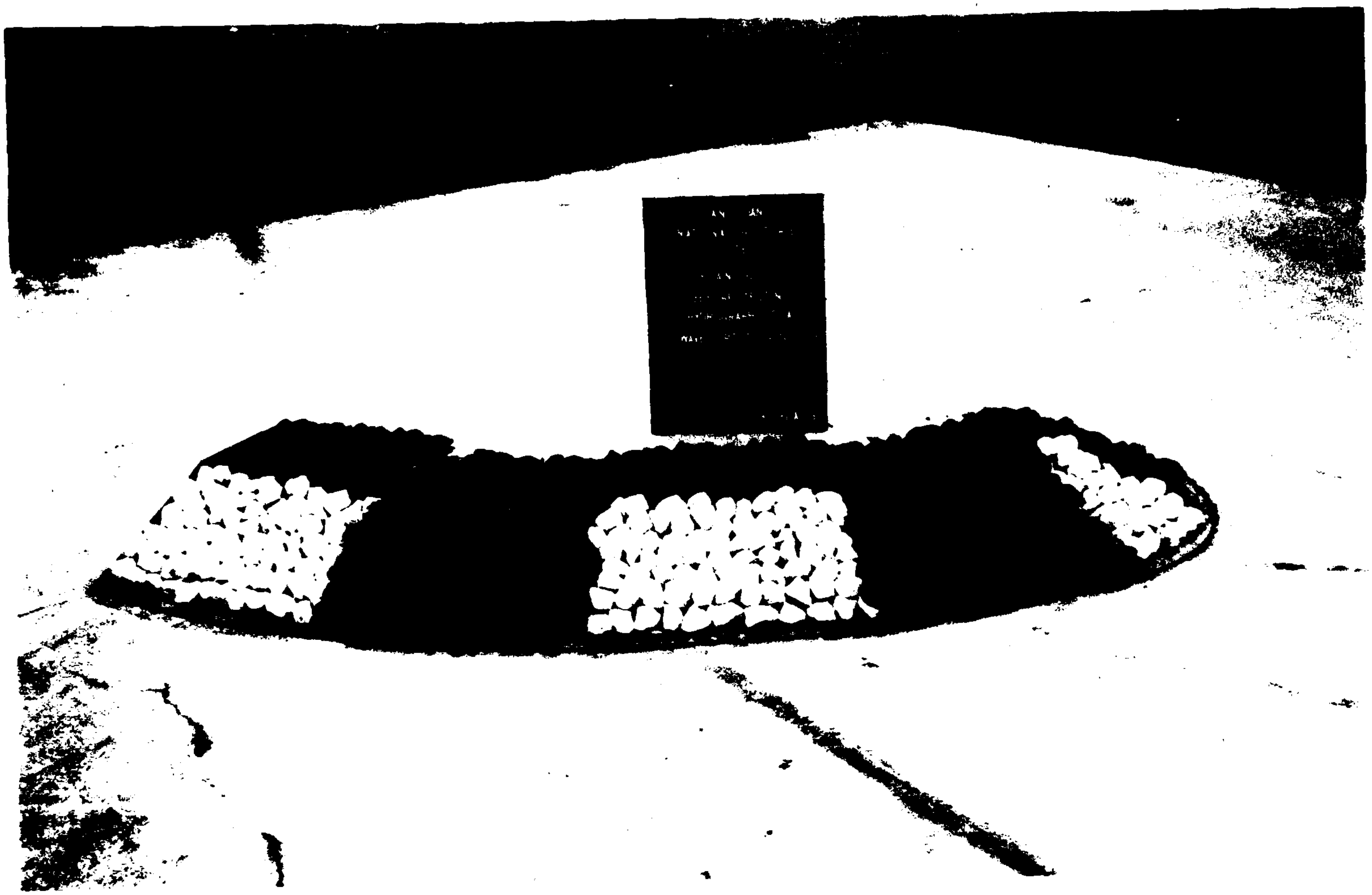

Photo 127. Sea-side view of Plan 3D-1 before testing north wave direction, 1st test section

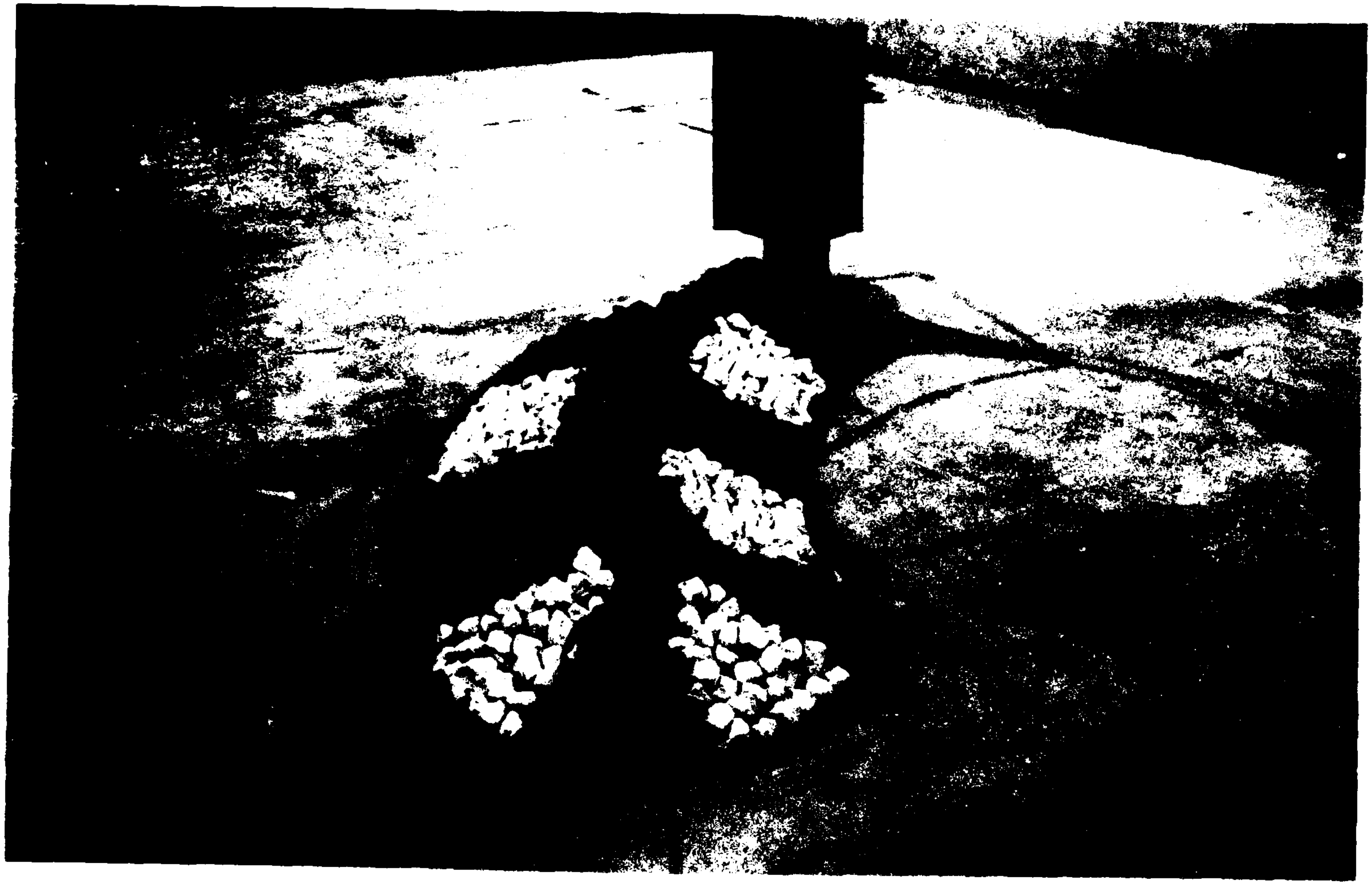

Photo 128. End view of Plan 3D-1 before testing north wave direction, 1st test section

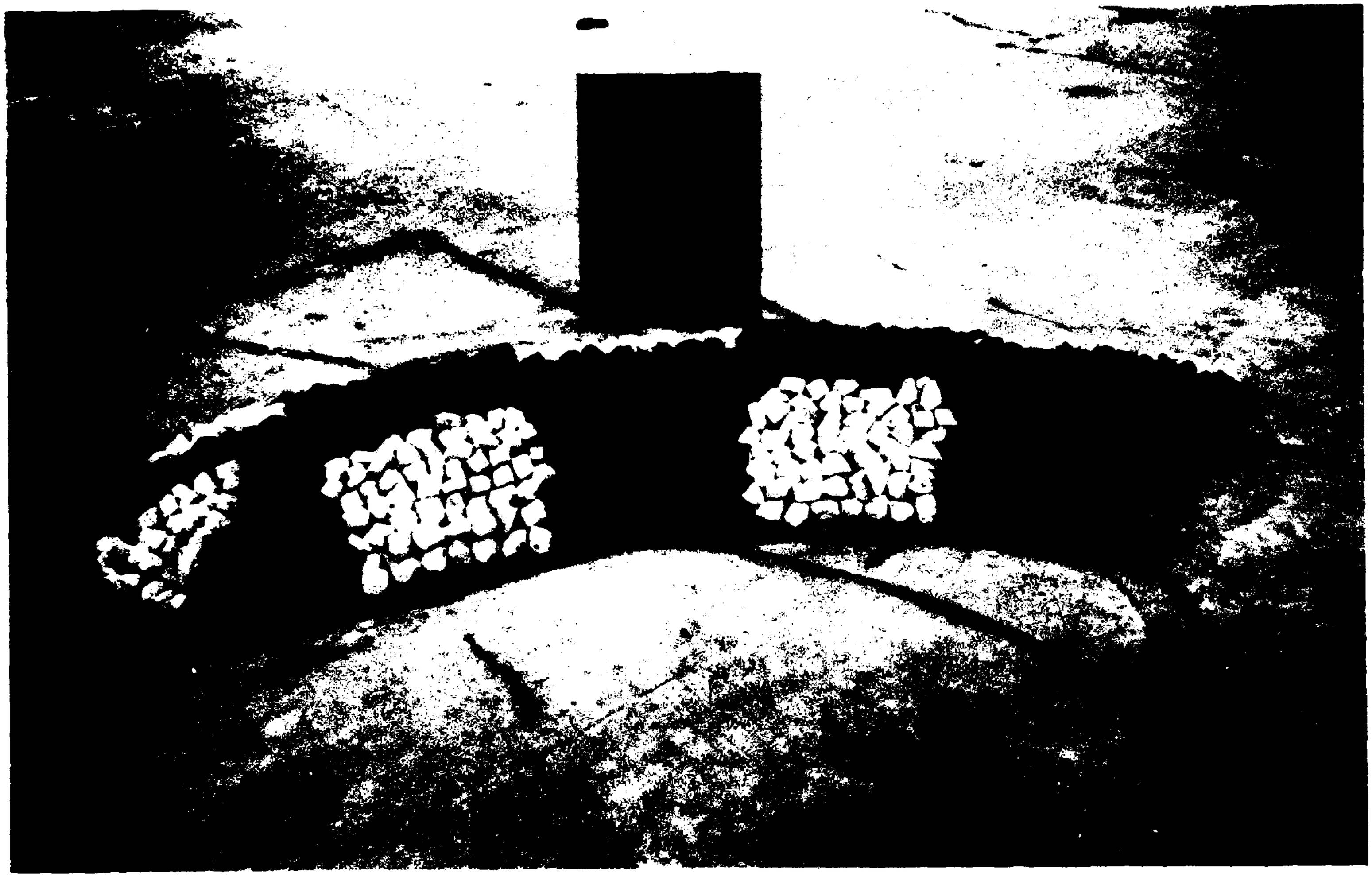

Photo 129. Beach-side view of Plan 3D-1 before testing north wave direction, 1st test section

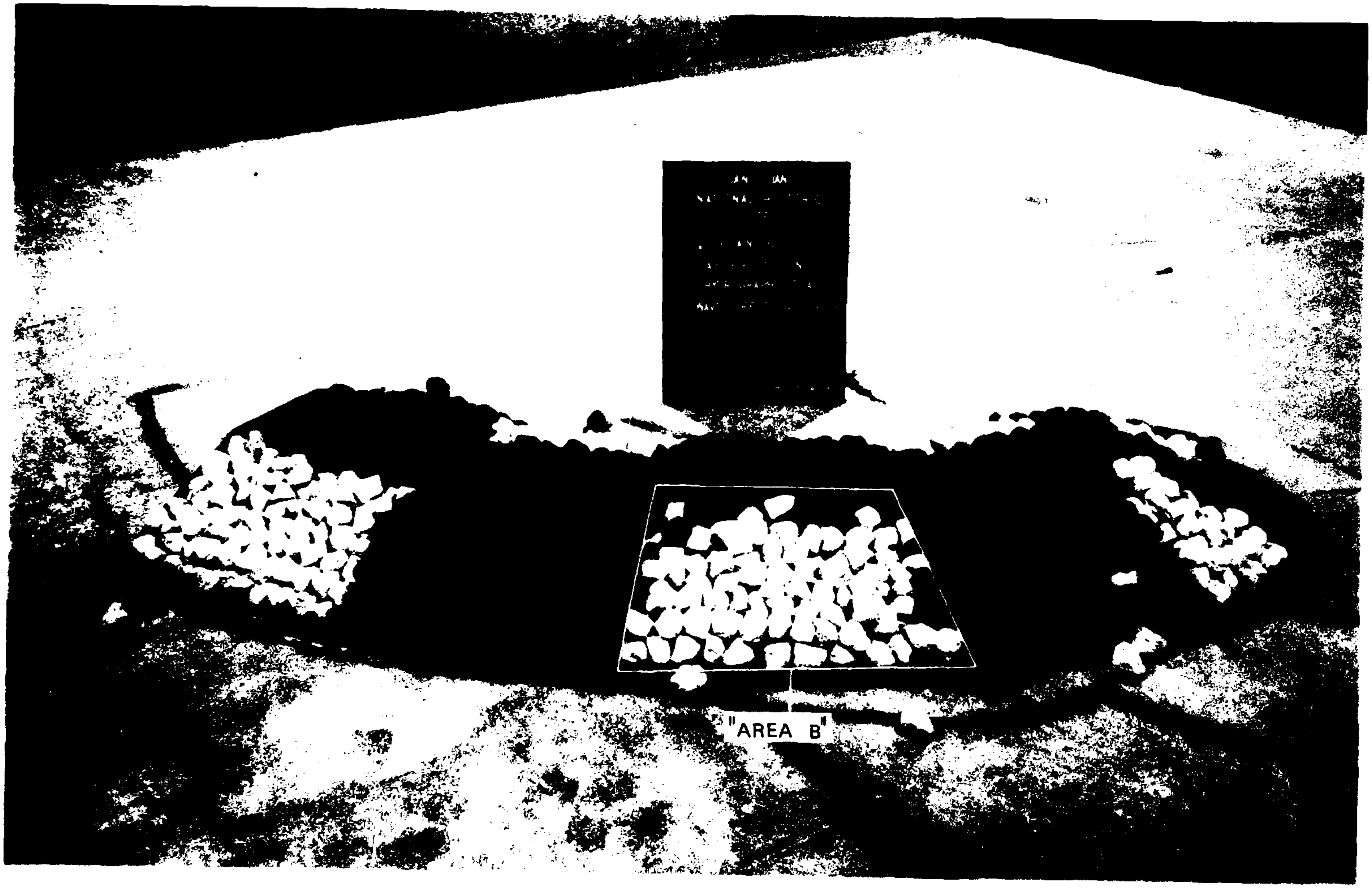

Photo 130. Sea-side view of Plan 3D-1 after testing Hydrograph 3D-A (Plate 3 and Table 3) for north wave direction, 1st test section

Photo 131. End view of Plan 3D-1 after testing Hydrograph 3D-A (Plate 3 and Table 3) for north wave direction, 1st test section

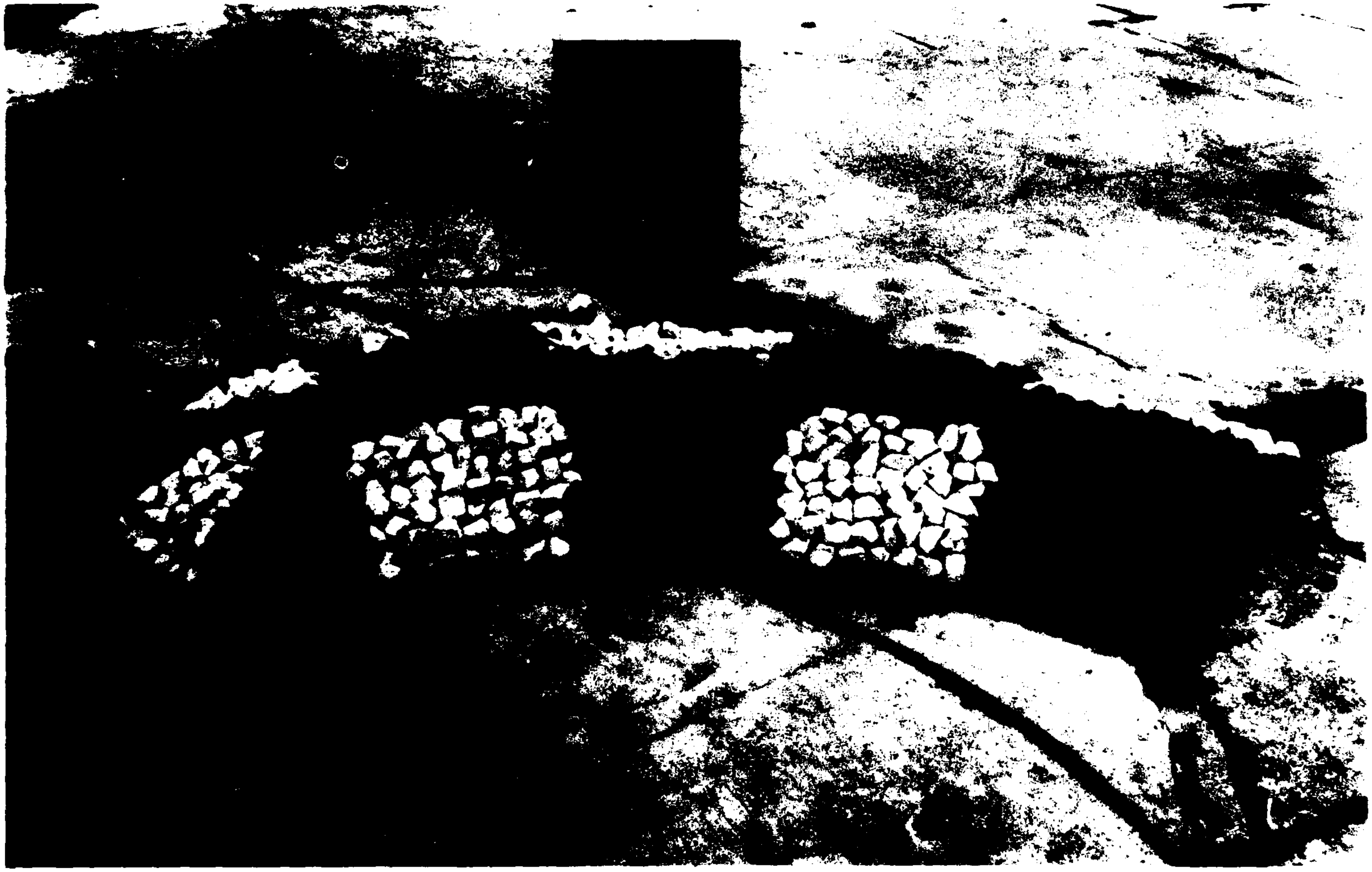

Photo 132. Beach-side view of Plan 3D-1 after testing Hydrograph 3D-A (Plate 3 and Table 3) for north wave direction, 1st test section

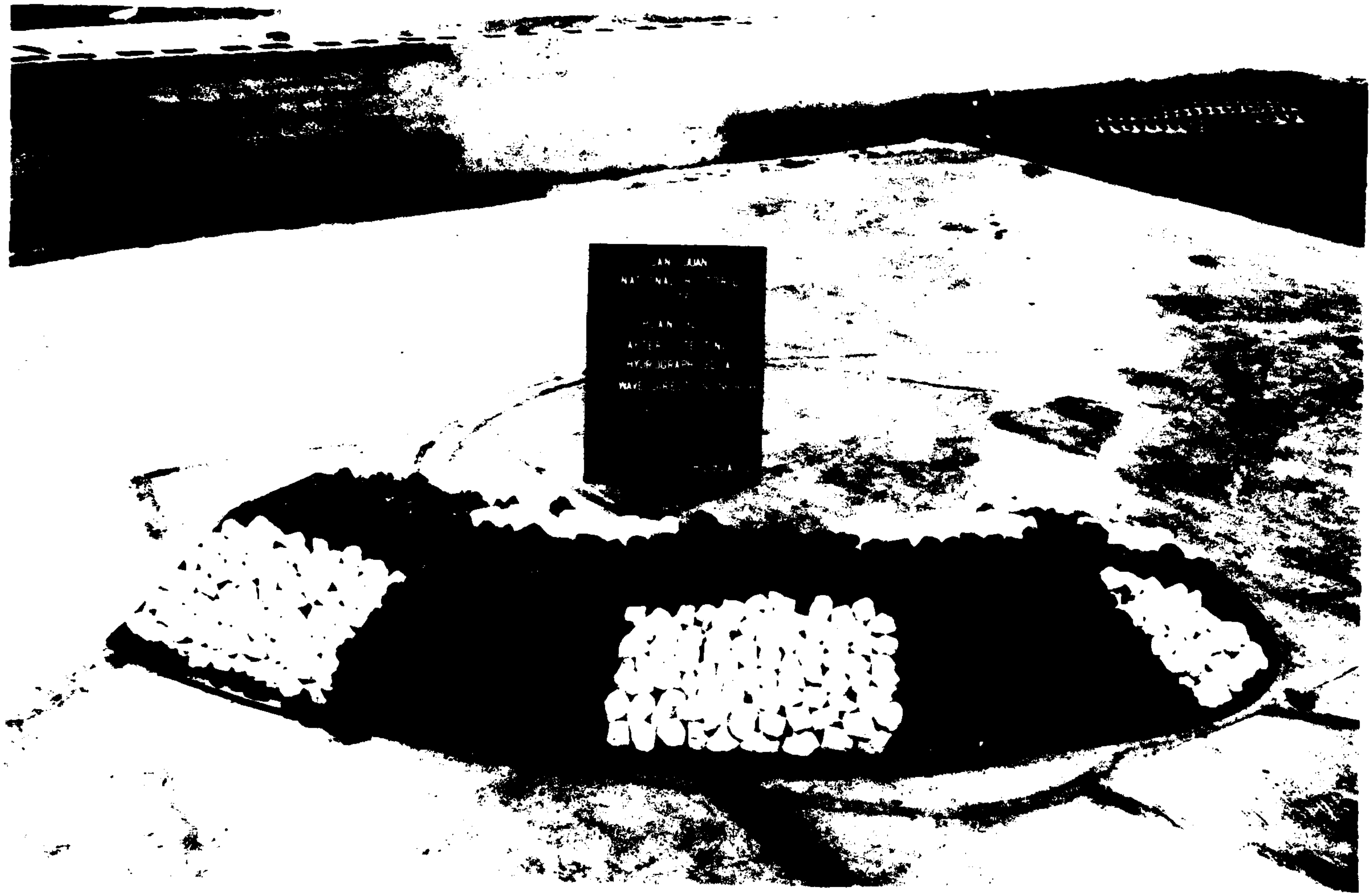

Photo 133. Sea-side view of Plan 3D-1 after testing Hydrograph 3D-A (Plate 3 and Table 3) for north wave direction, 2nd test section

Photo 134. End view of Plan 3D-1 after testing Hydrograph 3D-A (Plate 3 and Table 3) for north wave direction, 2nd test section

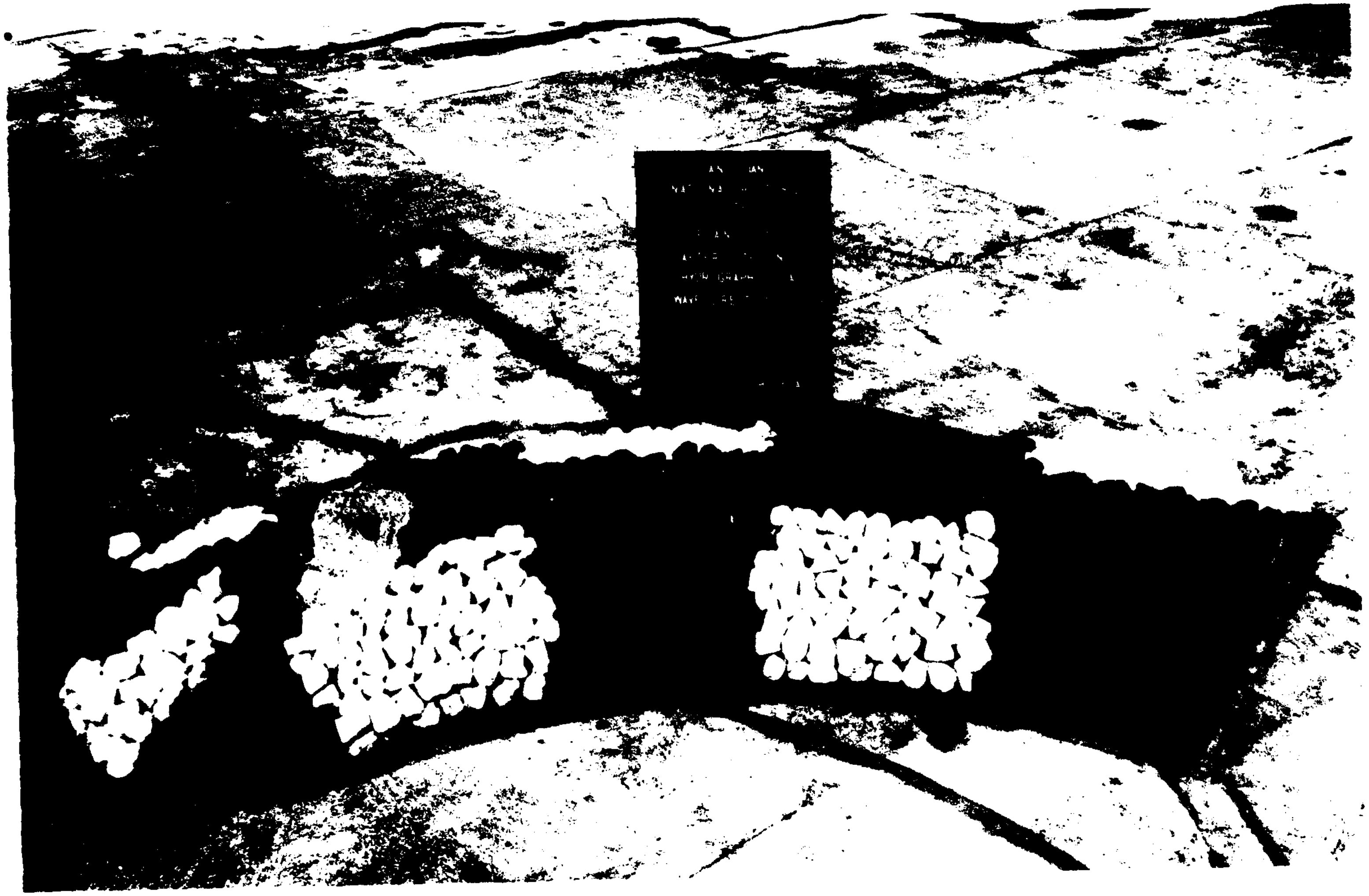

Photo 135. Beach-side view of Plan 3D-1 after testing Hydrograph 3D-A (Plate 3 and Table 3) for north wave direction, 2nd test section

Photo 136. Sea-side view of Plan 3D-1 before testing N72°W wave direction, 1st test section

Photo 137. End view of Plan 3D-1 before testing N72°W wave direction, 1st test section

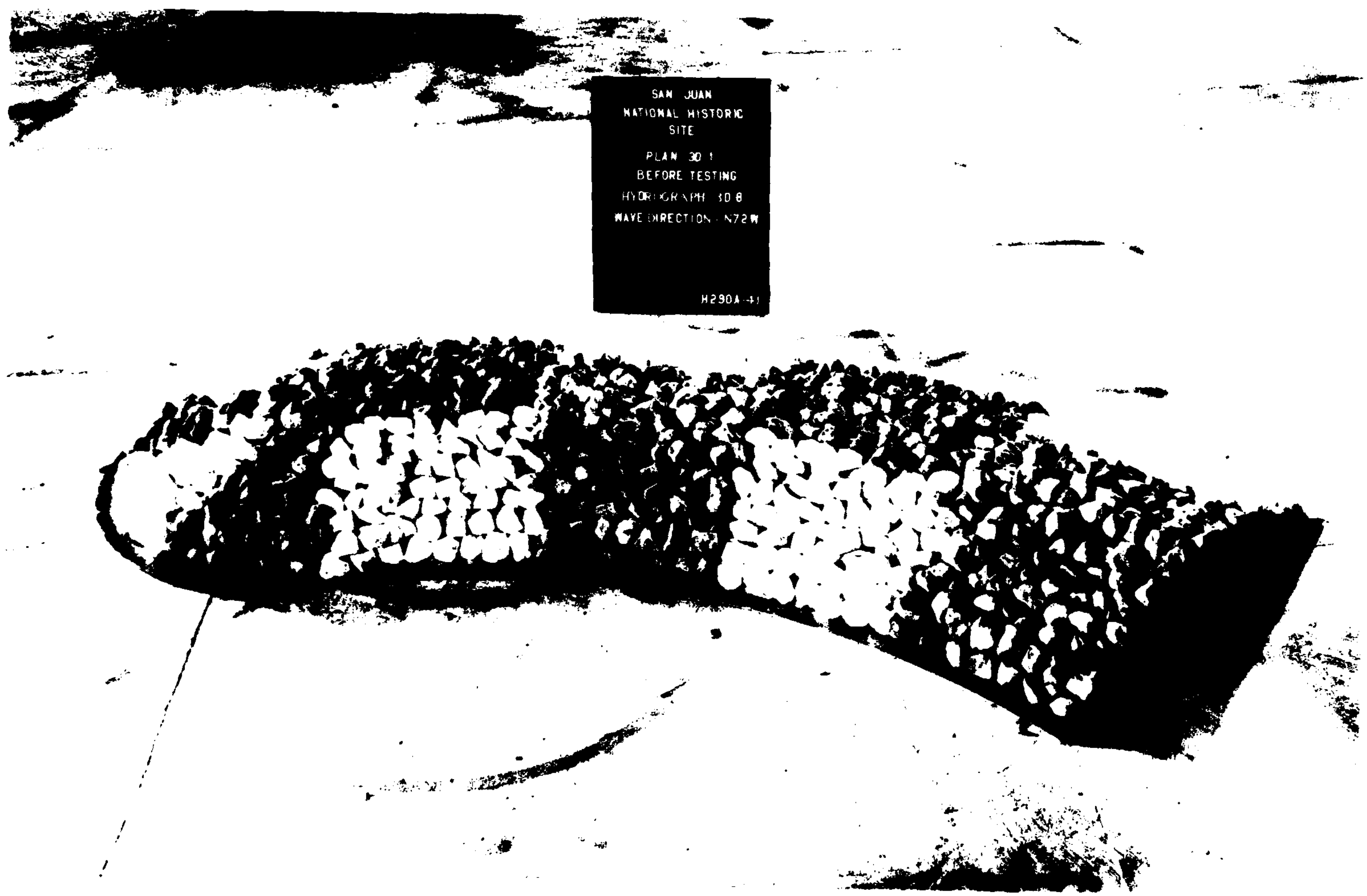

Photo 138. Beach-side view of Plan 3D-1 before testing N72°W wave direction, 1st test section

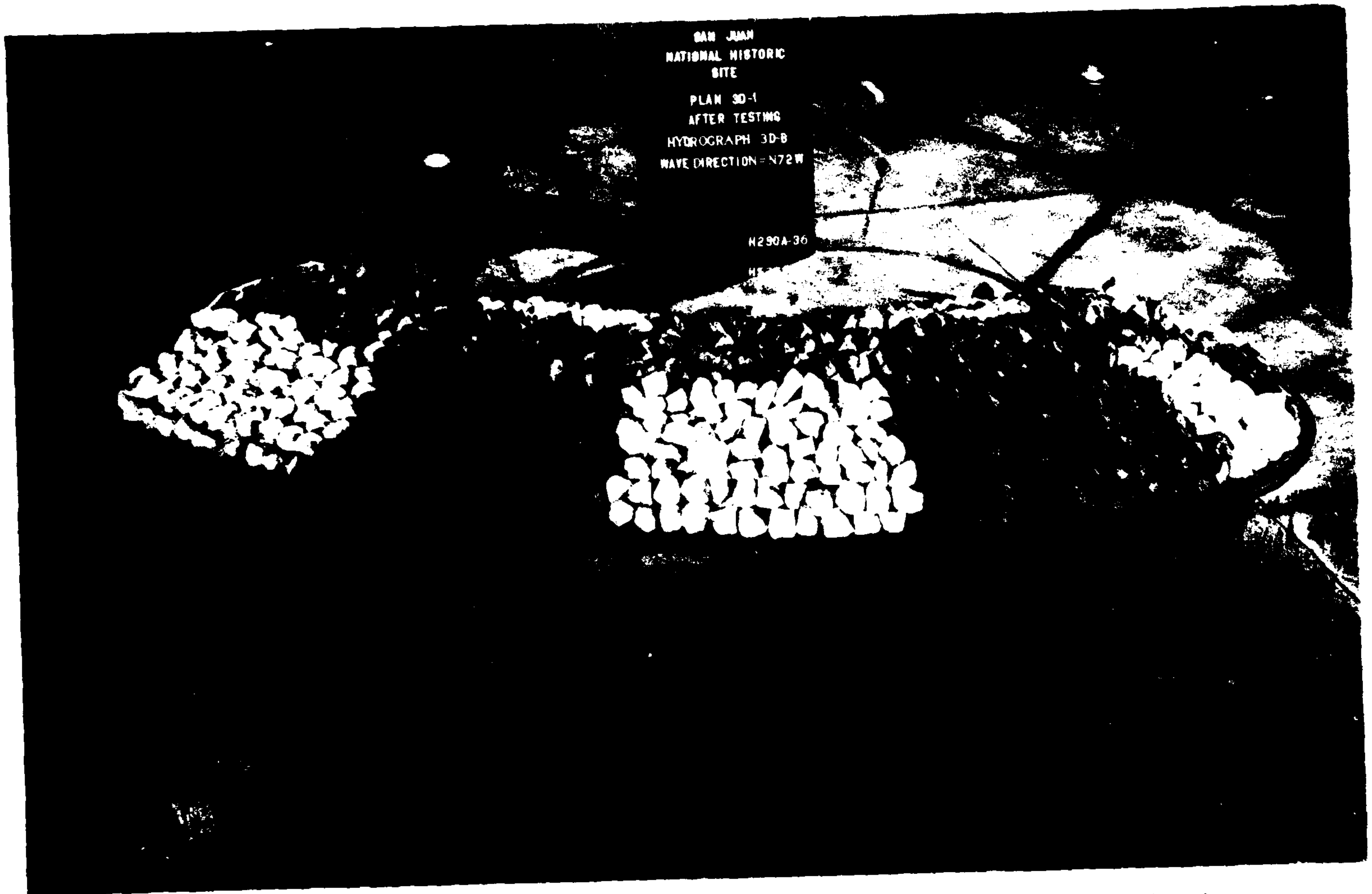

Photo 139. Sea-side view of Plan 3D-1 after testing Hydrograph 3D-B (Plate 4 and Table 4) for N72°W wave direction, 1st test section

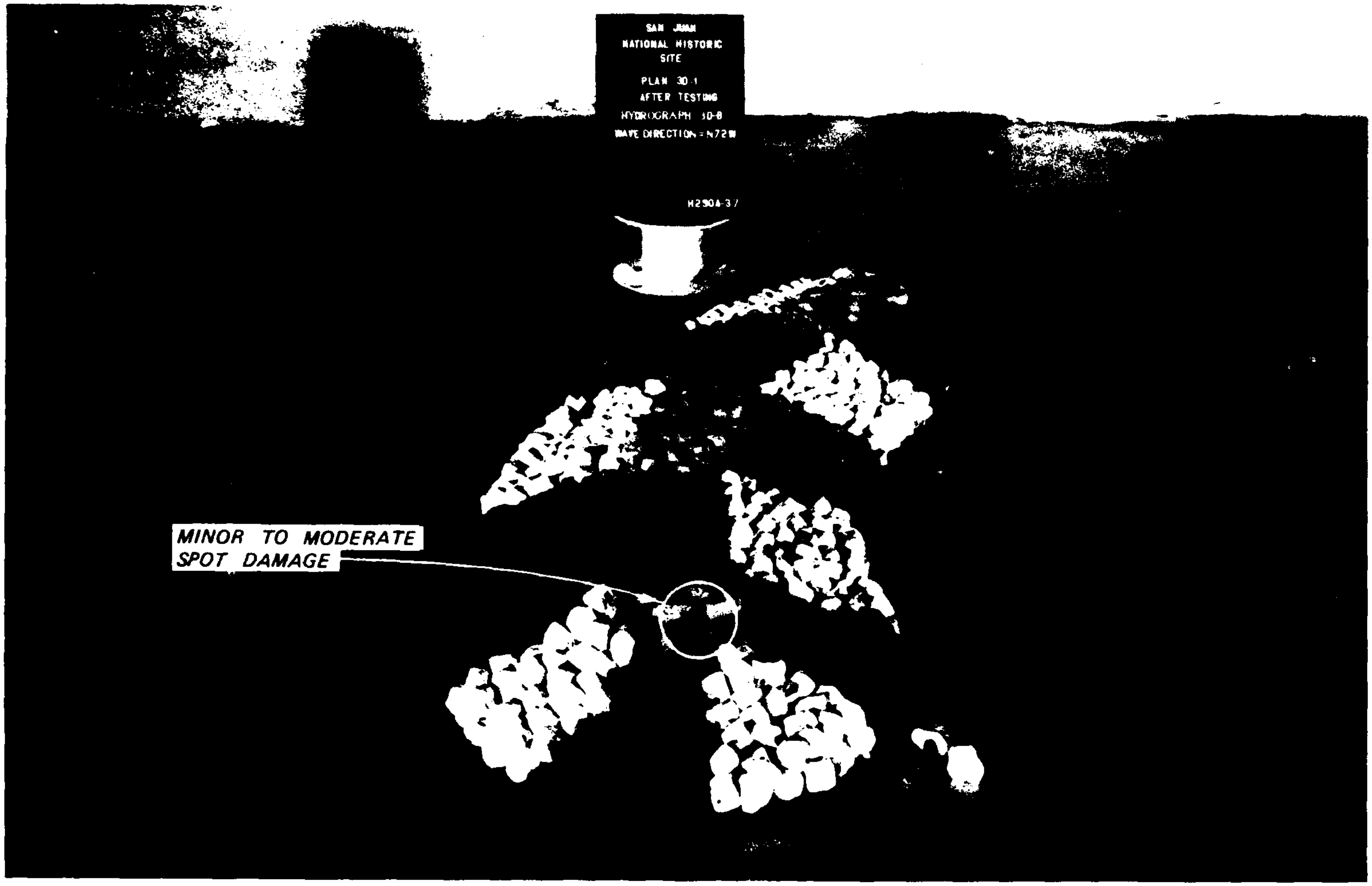

Photo 140. End view of Plan 3D-1 after testing Hydrograph 3D-B (Plate 4 and Table 4) for N72°W wave direction, 1st test section

AD-A108 120 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/2
BREAKWATER AND REVETMENT STABILITY STUDY, SAN JUAN NATIONAL HIS--ETC(U)
SEP 81 D G MARKLE
UNCLASSIFIED WES/TR/HL-81-11 NL

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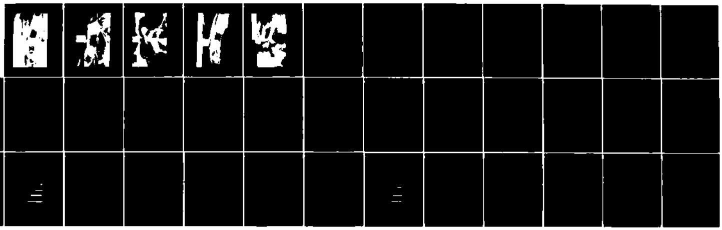

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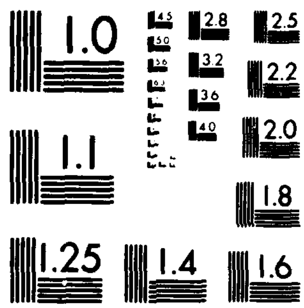


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS 1963 A

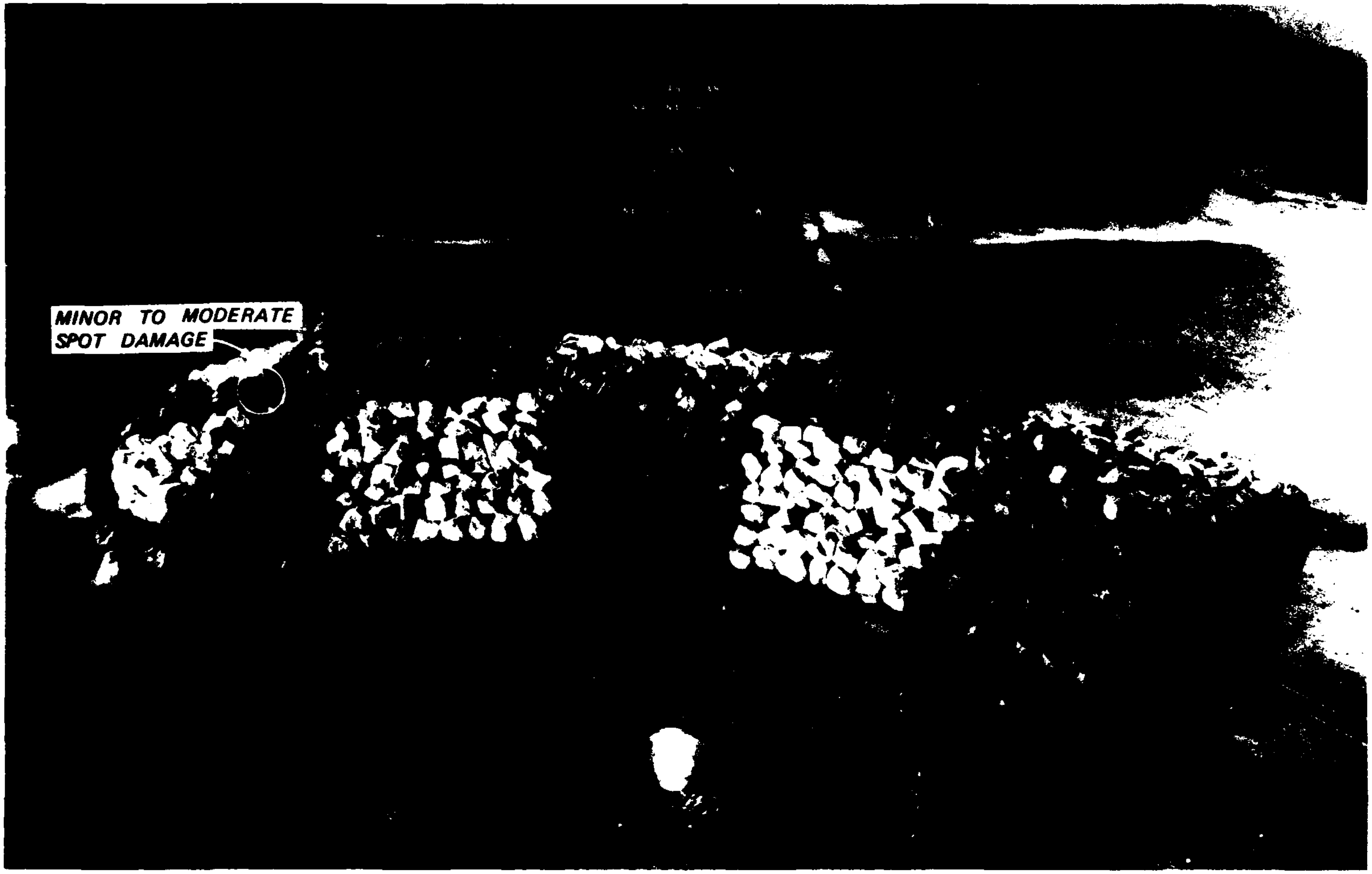


Photo 141. Beach-side view of Plan 3D-1 after testing Hydrograph 3D-B (Plate 4 and Table 4) for N72°W wave direction, 1st test section

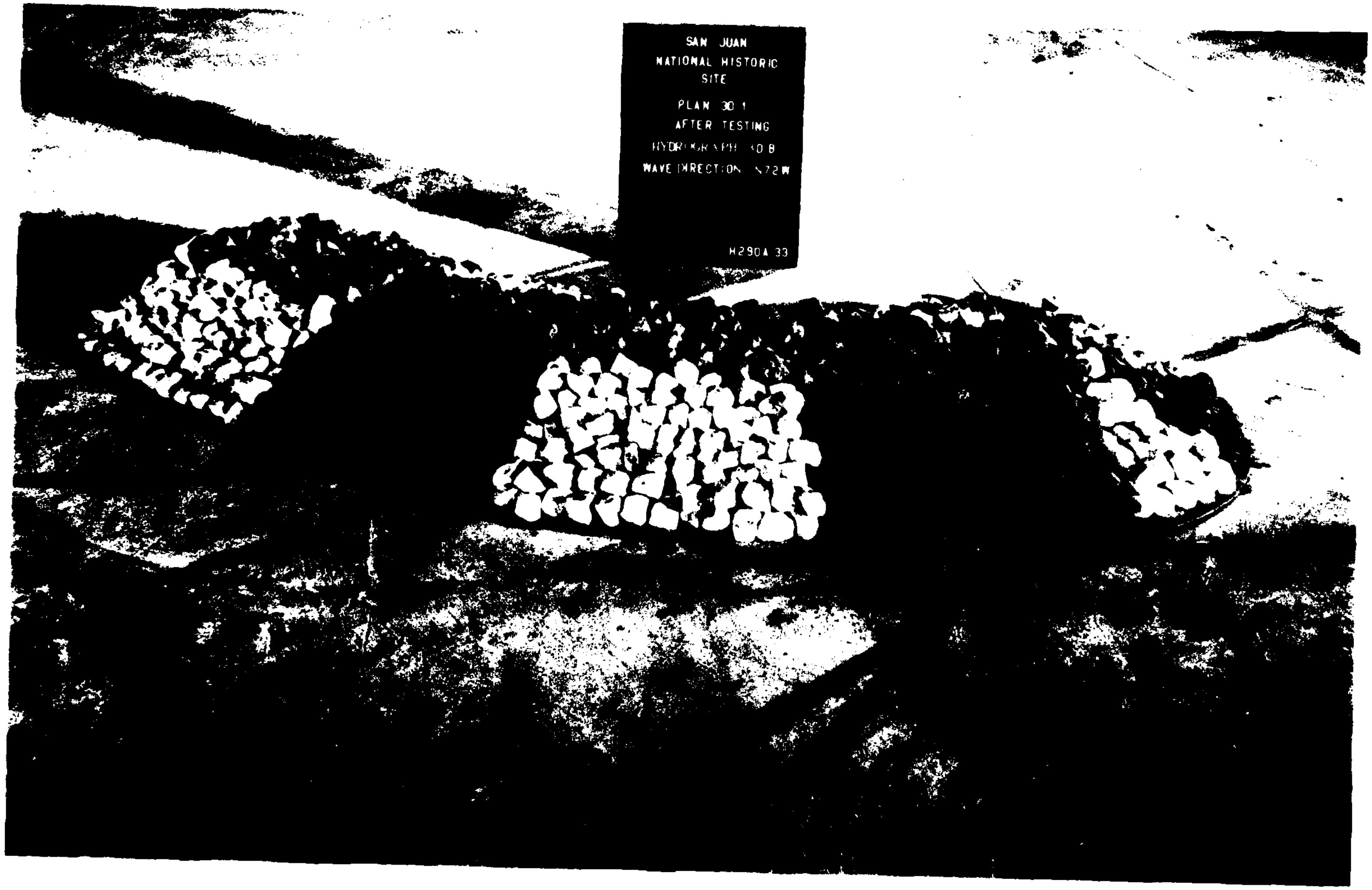


Photo 142. Sea-side view of Plan 3D-1 after testing Hydrograph 3D-B (Plate 4 and Table 4) for N72°W wave direction, 2nd test section

Photo 143. End view of Plan 3D-1 after testing Hydrograph 3D-B (Plate 4 and Table 4) for N72°W wave direction, 2nd test section

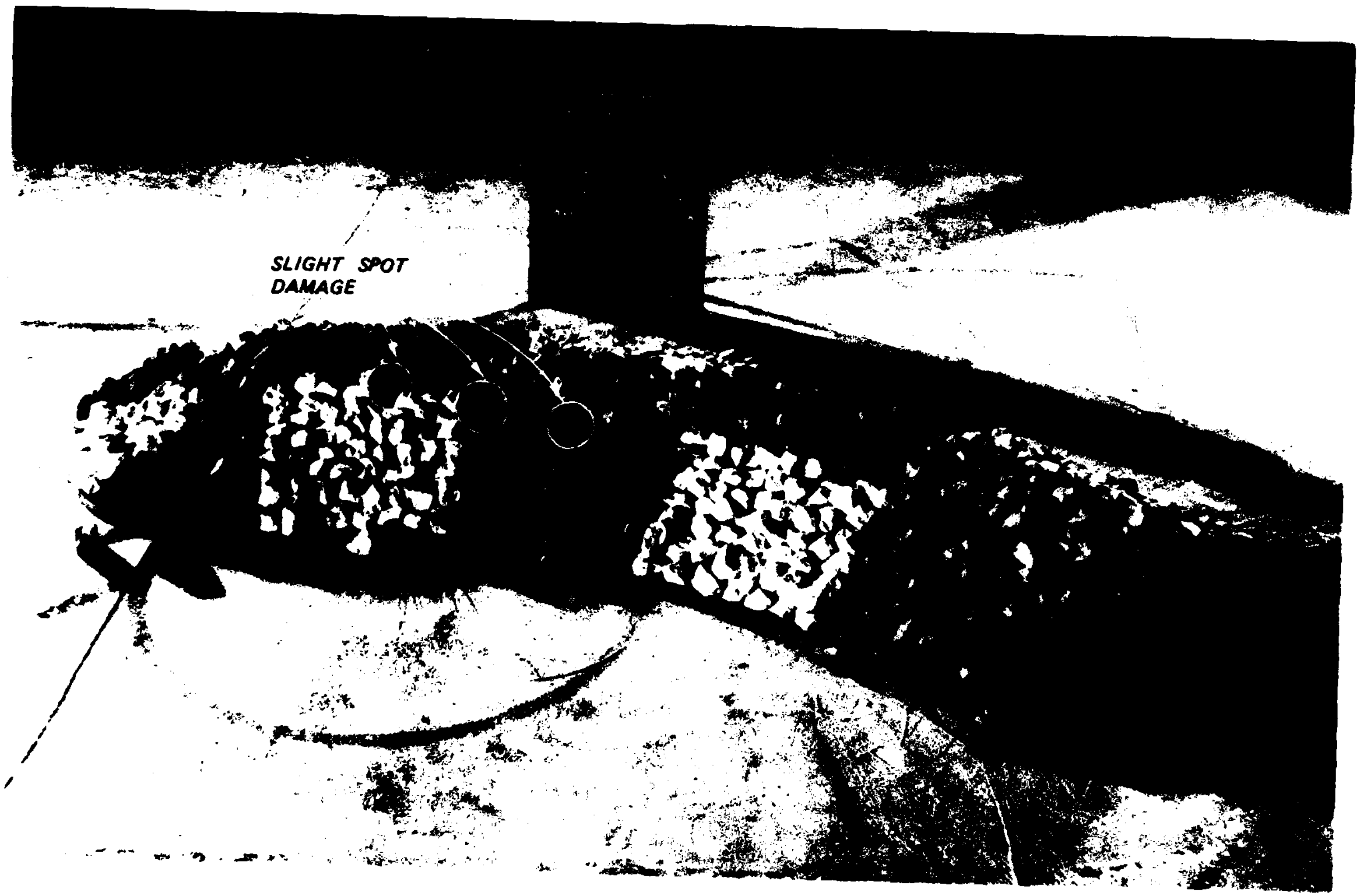
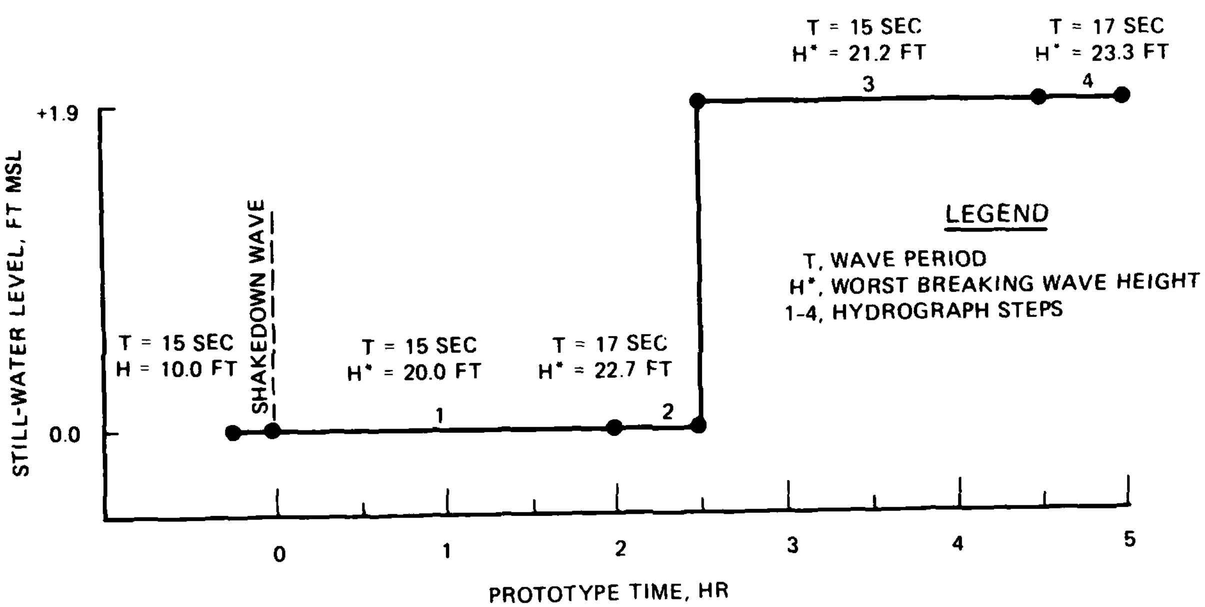


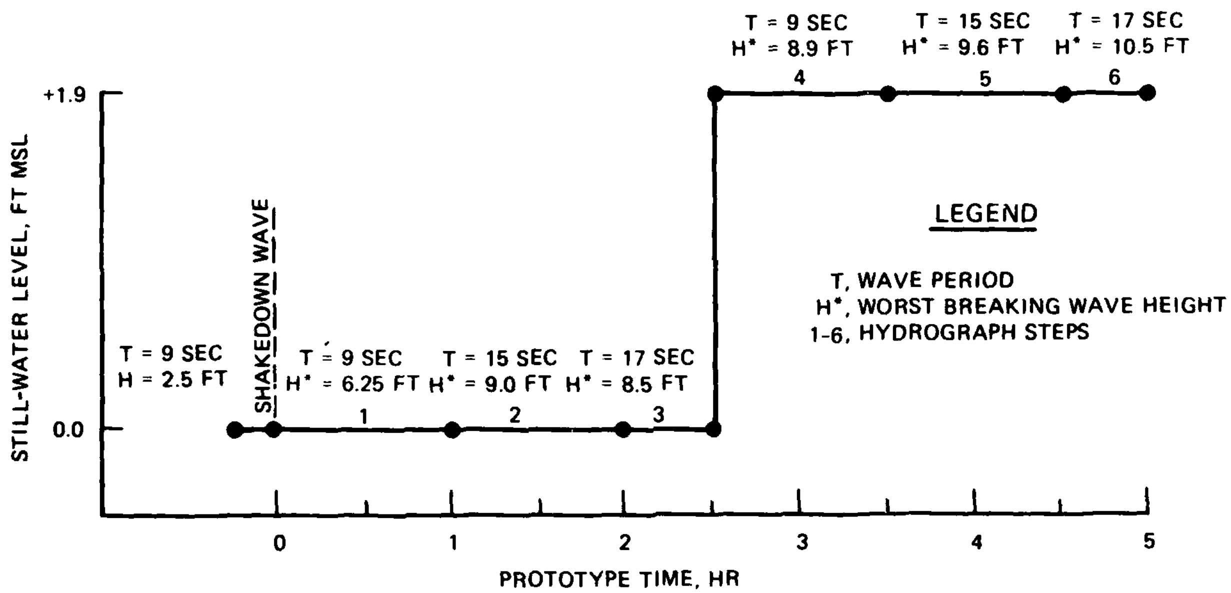
Photo 144. Beach-side view of Plan 3D-1 after testing Hydrograph 3D-B (Plate 4 and Table 4) for N72°W wave direction, 2nd test section

Photo 145. Wave attack on Plan 3D-1 for incident waves from N72°W, 0.0 swl



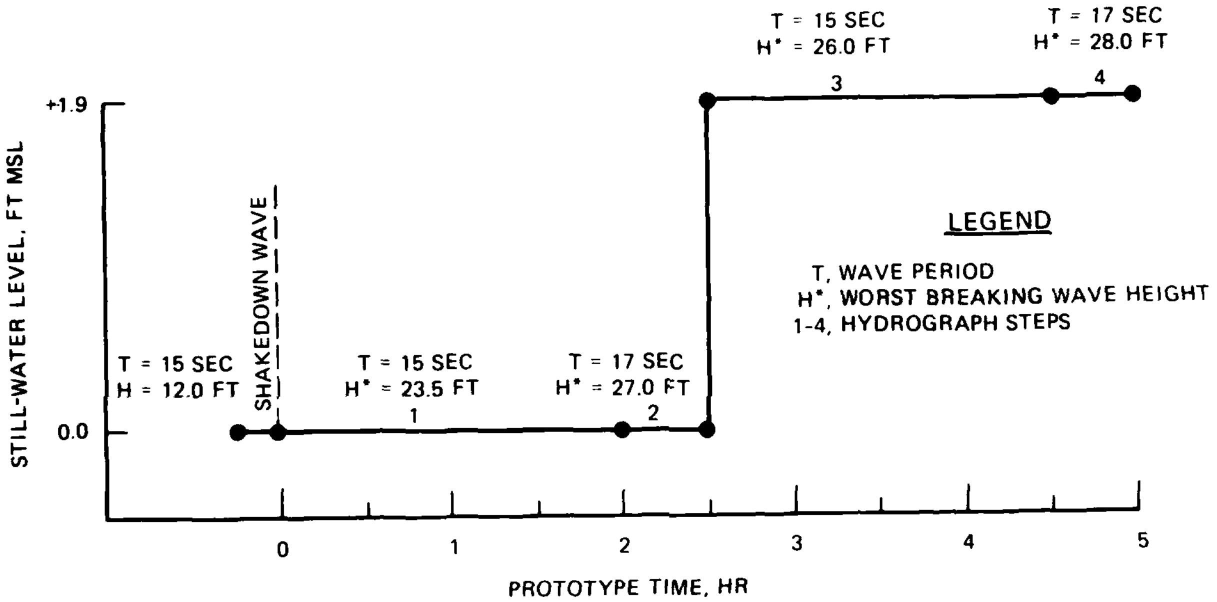
HYDROGRAPH A
2-D MODEL
NORTH SLOPE

PLATE 1



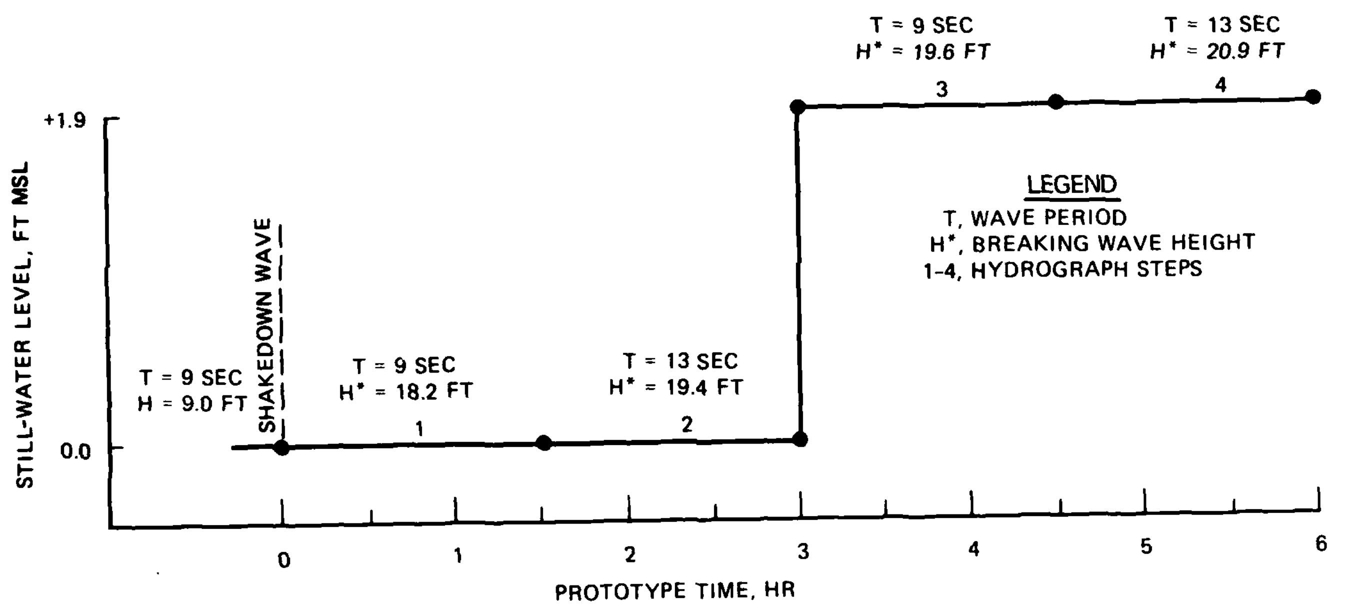
HYDROGRAPH B
2-D MODEL
WEST SLOPE

PLATE 2



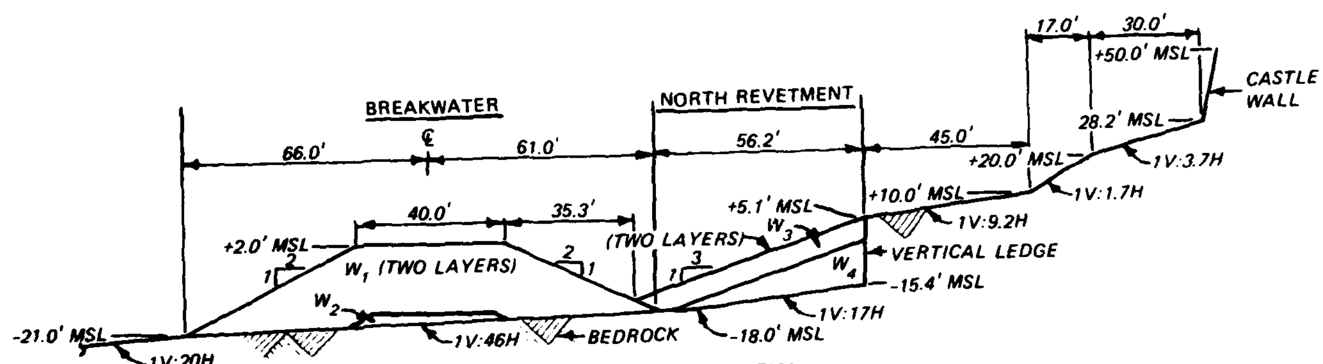
HYDROGRAPH 3D-A
3-D MODEL

PLATE 3



HYDROGRAPH 3D-B
3-D MODEL

PLATE 4

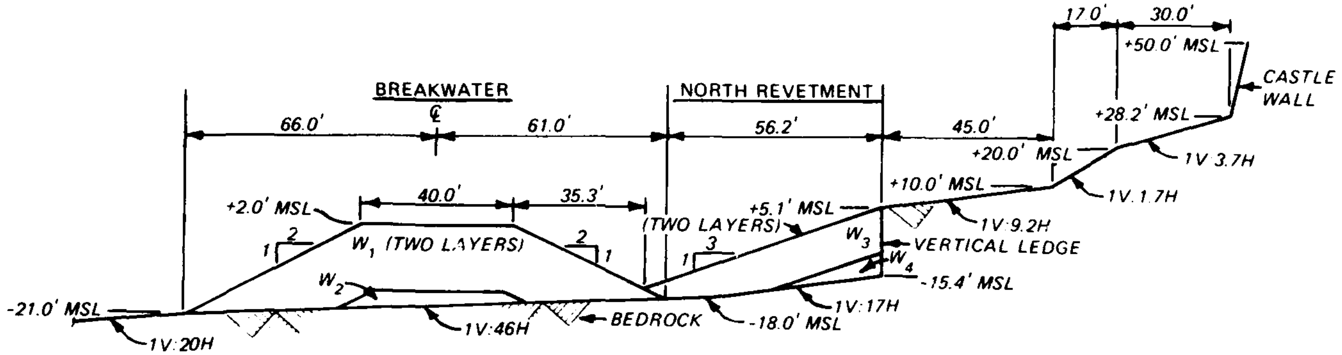


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 1.450-LB ROCK @ 165 PCF	W_1 = 93,600-LB ROCK @ 165 PCF
W_2 = 0.073-LB ROCK @ 165 PCF	W_2 = 4,680-LB ROCK @ 165 PCF
W_3 = 0.033-LB ROCK @ 165 PCF	W_3 = 2,100-LB ROCK @ 165 PCF
W_4 = 0.003-LB ROCK @ 165 PCF	W_4 = 210-LB ROCK @ 165 PCF

NORTH SLOPE
PLAN N-1

PLATE 5



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 1.050-LB ROCK @ 165 PCF	W_1 = 67,800-LB ROCK @ 165 PCF
W_2 = 0.053-LB ROCK @ 165 PCF	W_2 = 3,390-LB ROCK @ 165 PCF
W_3 = 0.210-LB ROCK @ 165 PCF	W_3 = 13,550-LB ROCK @ 165 PCF
W_4 = 0.021-LB ROCK @ 165 PCF	W_4 = 1,360-LB ROCK @ 165 PCF

NORTH SLOPE
PLAN N-2

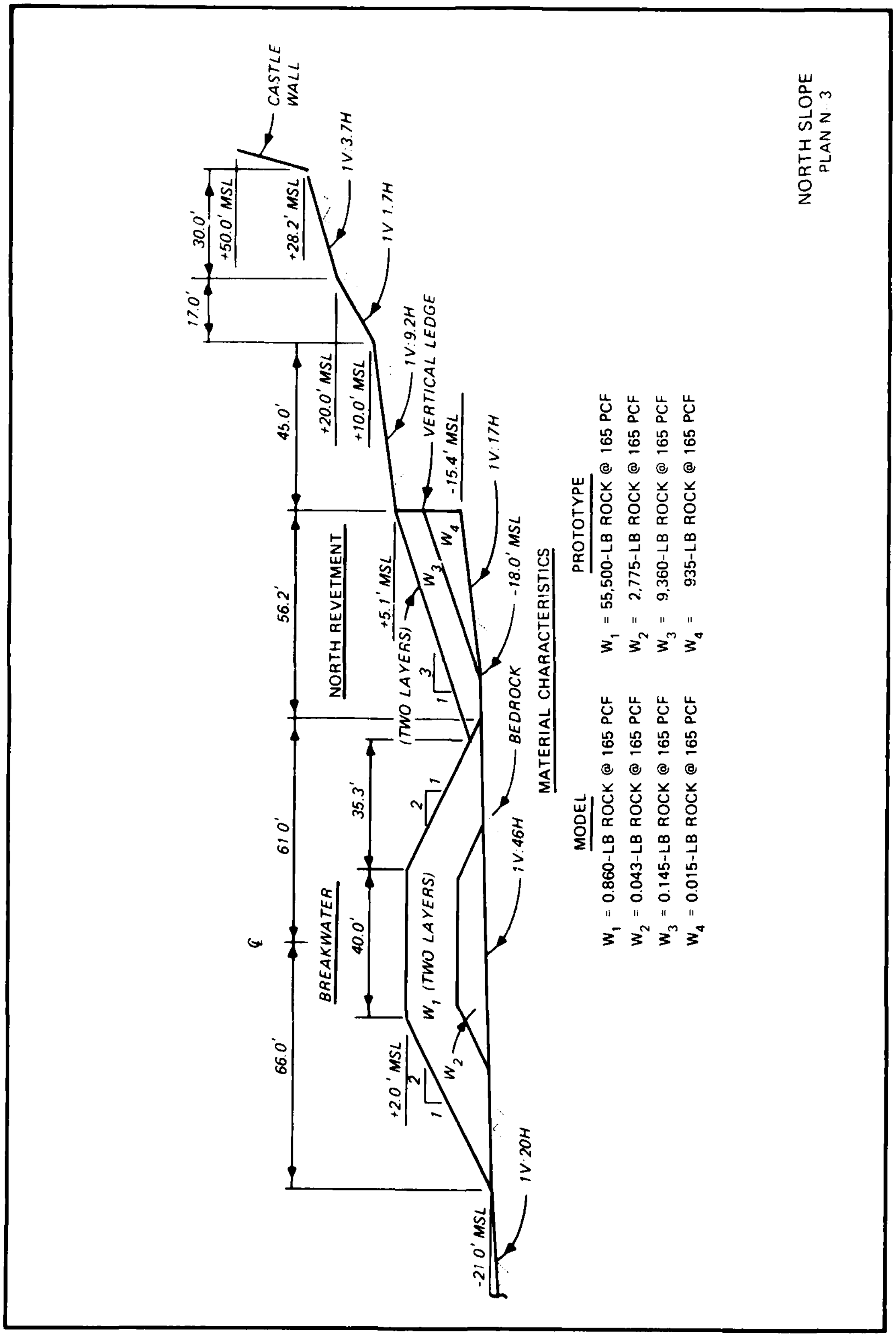
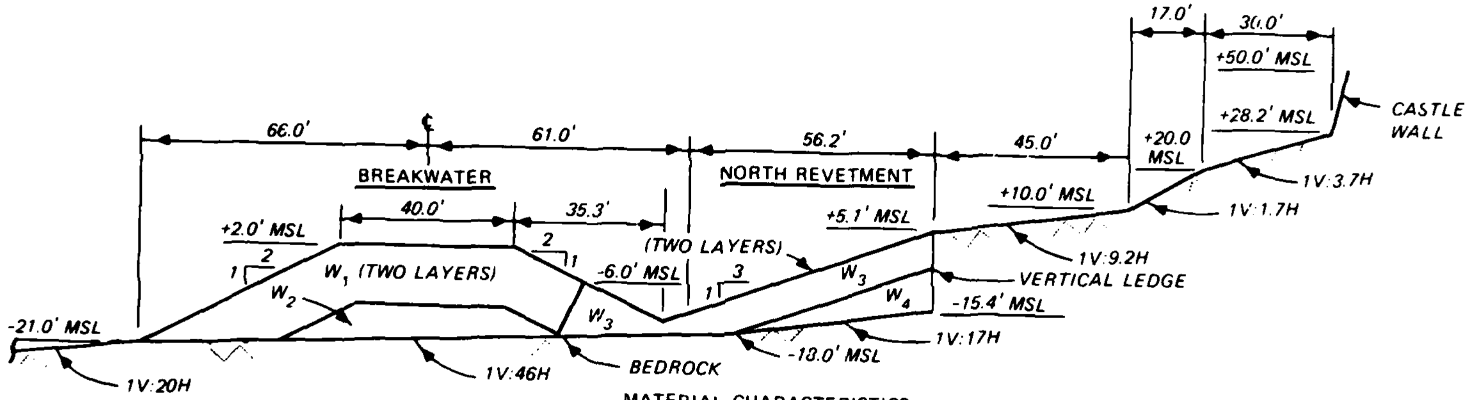

CASTLE WALL
1V:3.7H
1V:1.7H
1V:9.2H
VERTICAL LEDGE
1V:17H
30.0'
+50.0' MSL
+28.2' MSL
17.0'
45.0'
+20.0' MSL
+10.0' MSL
-15.4' MSL
56.2'
NORTH REVETMENT
+5.1' MSL
W_3
W_4
-18.0' MSL
(TWO LAYERS)
BEDROCK
61.0'
35.3'
40.0'
BREAKWATER
W_1 (TWO LAYERS)
1V:46H
66.0'
+2.0' MSL
W_2
1V:20H
-21.0' MSL
MATERIAL CHARACTERISTICS
MODEL
W_1 = 0.860-LB ROCK @ 165 PCF
W_2 = 0.043-LB ROCK @ 165 PCF
W_3 = 0.145-LB ROCK @ 165 PCF
W_4 = 0.015-LB ROCK @ 165 PCF
PROTOTYPE
W_1 = 55,500-LB ROCK @ 165 PCF
W_2 = 2,775-LB ROCK @ 165 PCF
W_3 = 9,360-LB ROCK @ 165 PCF
W_4 = 935-LB ROCK @ 165 PCF
NORTH SLOPE
PLAN N-3

PLATE 7

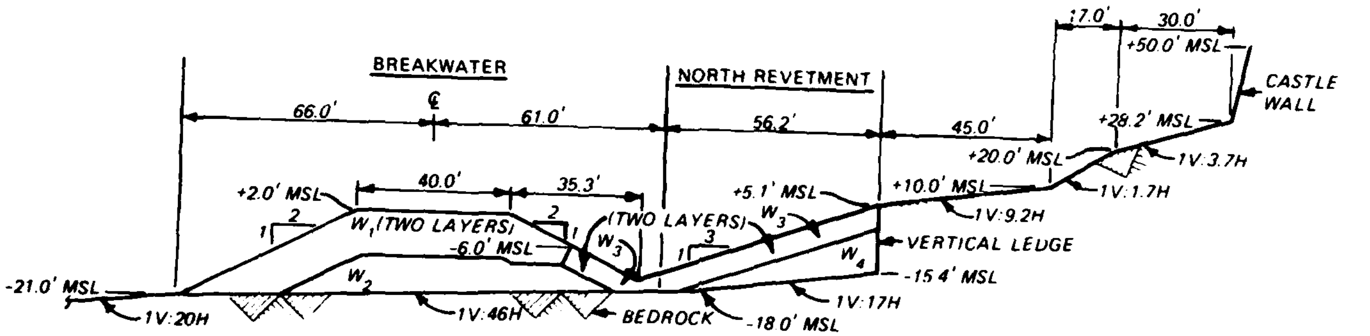


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.860-LB ROCK @ 165 PCF	W_1 = 55,500-LB ROCK @ 165 PCF
W_2 = 0.043-LB ROCK @ 165 PCF	W_2 = 2,775-LB ROCK @ 165 PCF
W_3 = 0.145-LB ROCK @ 165 PCF	W_3 = 9,360-LB ROCK @ 165 PCF
W_4 0.015-LB ROCK @ 165 PCF	W_4 = 935-LB ROCK @ 165 PCF

NORTH SLOPE
PLAN N-3-A

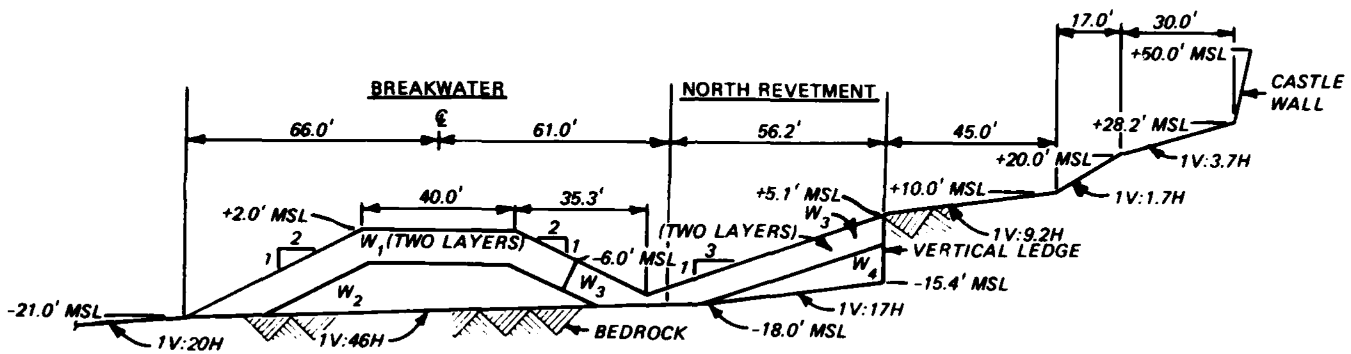
PLATE 8



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.71-LB ROCK @ 165 PCF	W_1 = 45,825-LB ROCK @ 165 PCF
W_2 = 0.036-LB ROCK @ 165 PCF	W_2 = 2,290-LB ROCK @ 165 PCF
W_3 = 0.095-LB ROCK @ 165 PCF	W_3 = 6,100-LB ROCK @ 165 PCF
W_4 = 0.010-LB ROCK @ 165 PCF	W_4 = 610-LB ROCK @ 165 PCF

NORTH SLOPE
PLAN N-4

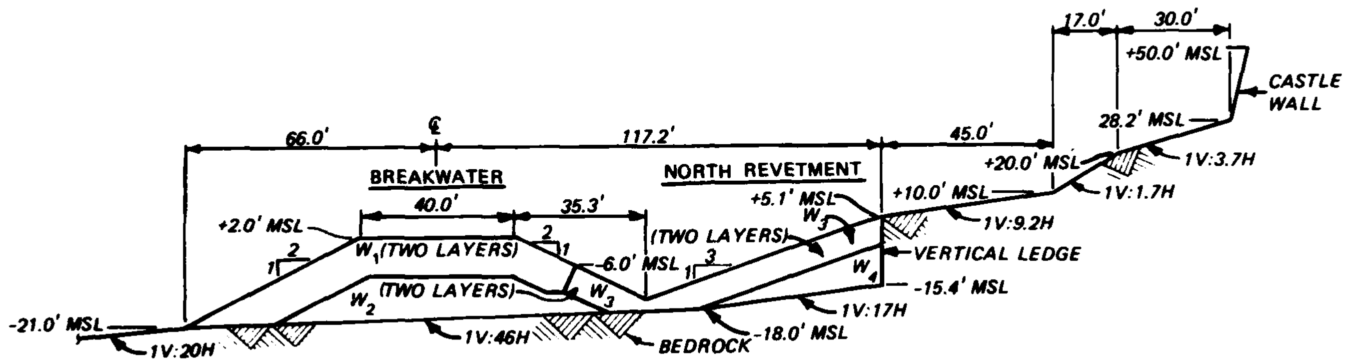


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.298-LB DOLOSSE @ 141.5 PCF	W_1 = 15,135-LB DOLOSSE @ 150 PCF
W_2 = 0.047-LB ROCK @ 165 PCF	W_2 = 3,027-LB ROCK @ 165 PCF
W_3 = 0.145-LB ROCK @ 165 PCF	W_3 = 9,360-LB ROCK @ 165 PCF
W_4 = 0.015-LB ROCK @ 165 PCF	W_4 = 935-LB ROCK @ 165 PCF

NORTH SLOPE
PLAN N-5

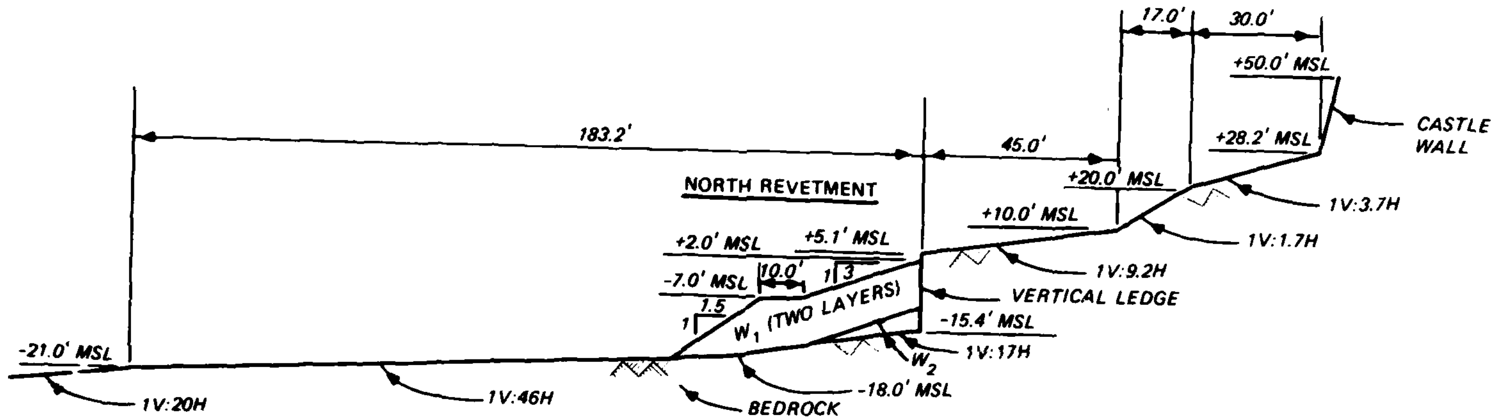
PLATE 10



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.442-LB DOLOSSE @ 140.6 PCF	W_1 = 21,830-LB DOLOSSE @ 150 PCF
W_2 = 0.068-LB ROCK @ 165 PCF	W_2 = 4,366-LB ROCK @ 165 PCF
W_3 = 0.145-LB ROCK @ 165 PCF	W_3 = 9,360-LB ROCK @ 165 PCF
W_4 = 0.015-LB ROCK @ 165 PCF	W_4 = 935-LB ROCK @ 165 PCF

NORTH SLOPE
PLAN N-6



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.38-LB ROCK @ 165 PCF	W_1 = 24,530-LB ROCK @ 165 PCF
W_2 = 0.038-LB ROCK @ 165 PCF	W_2 = 2,453-LB ROCK @ 165 PCF

NORTH SLOPE
PLAN N-7

PLATE 12

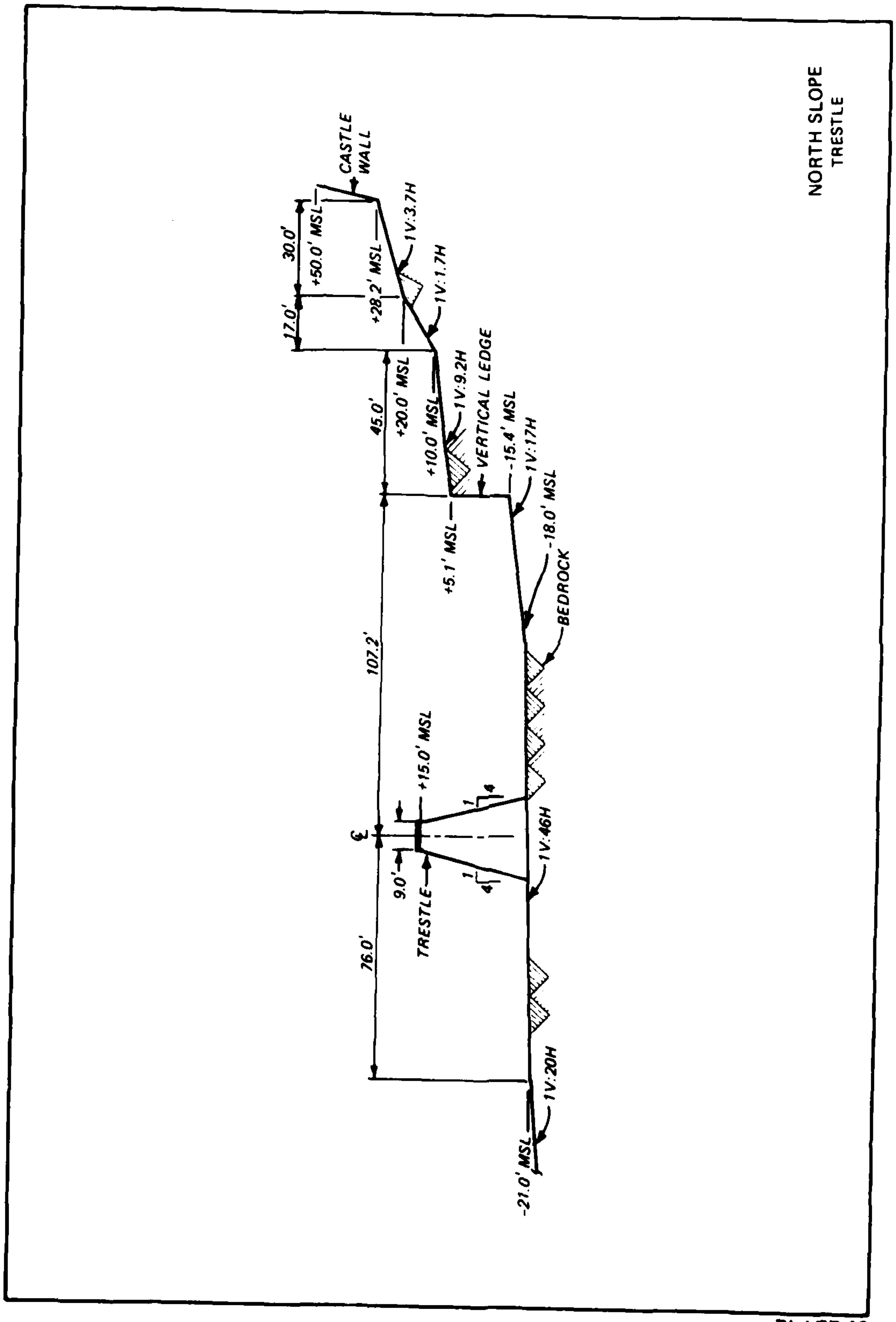
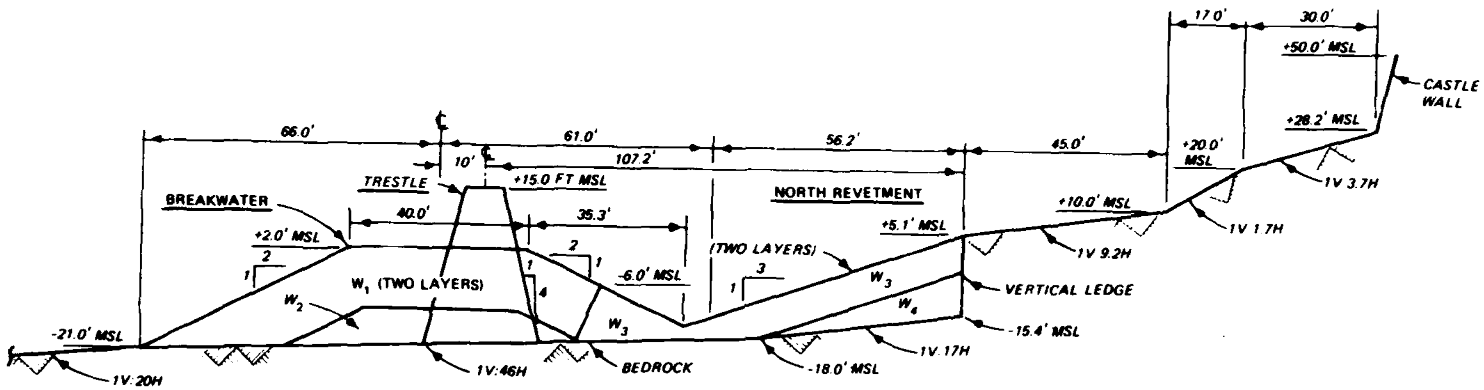

NORTH SLOPE
TRESTLE
CASTLE
WALL
30.0'
+50.0' MSL
1V:3.7H
+28.2' MSL
1V:1.7H
17.0'
45.0'
+20.0' MSL
1V:9.2H
+10.0' MSL
VERTICAL LEDGE
-15.4' MSL
1V:17H
+5.1' MSL
-18.0' MSL
BEDROCK
107.2'
+15.0' MSL
1
4
℄
9.0'
TRESTLE
1V:46H
76.0'
1V:20H
-21.0' MSL

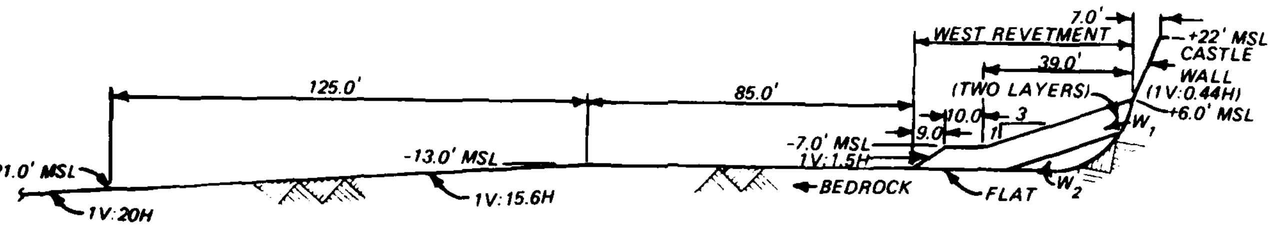
PLATE 13



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.860-LB ROCK @ 165 PCF	W_1 = 55,500-LB ROCK @ 165 PCF
W_2 = 0.043-LB ROCK @ 165 PCF	W_2 = 2,775-LB ROCK @ 165 PCF
W_3 = 0 145-LB ROCK @ 165 PCF	W_3 = 9,380-LB ROCK @ 165 PCF
W_4 = 0.015-LB ROCK @ 165 PCF	W_4 = 935-LB ROCK @ 165 PCF

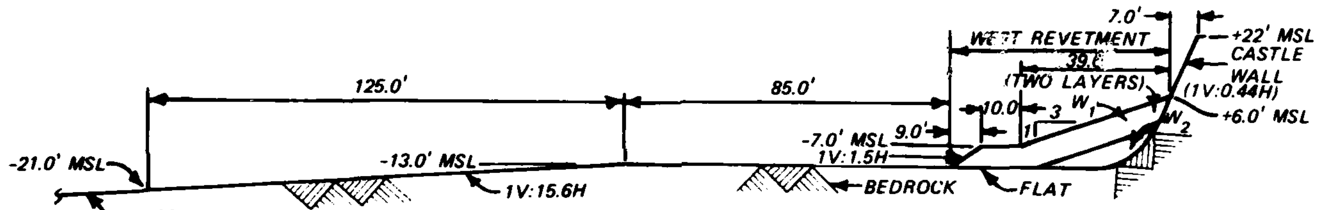
NORTH SLOPE
PLAN N-3-A AND TRESTLE



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.16-LB ROCK @ 165 PCF	W_1 = 10,330-LB ROCK @ 165 PCF
W_2 = 0.016-LB ROCK @ 165 PCF	W_2 = 1,033-LB ROCK @ 165 PCF

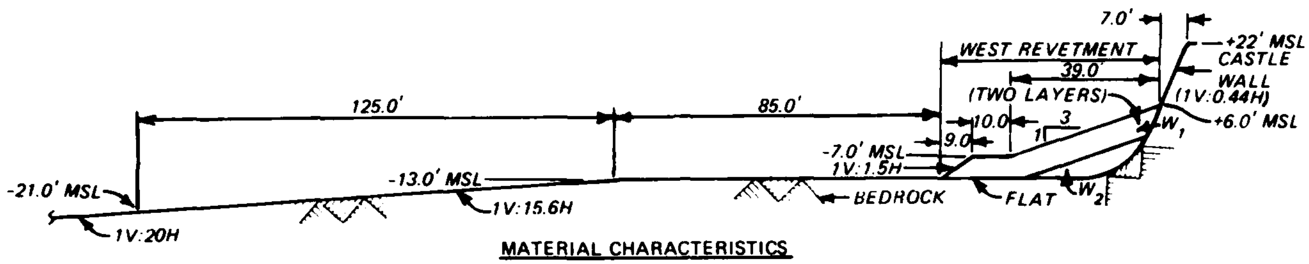
WEST SLOPE
PLAN W-1



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.109-LB ROCK @ 165 PCF	W_1 = 7,000-LB ROCK @ 165 PCF
W_2 = 0.011-LB ROCK @ 165 PCF	W_2 = 700-LB ROCK @ 165 PCF

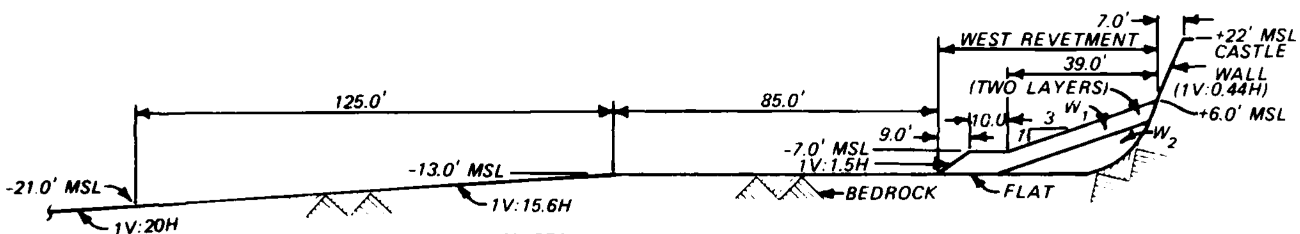
WEST SLOPE
PLAN W-2



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.062-LB ROCK @ 165 PCF	W_1 = 4,000-LB ROCK @ 165 PCF
W_2 = 0.006-LB ROCK @ 165 PCF	W_2 = 400-LB ROCK @ 165 PCF

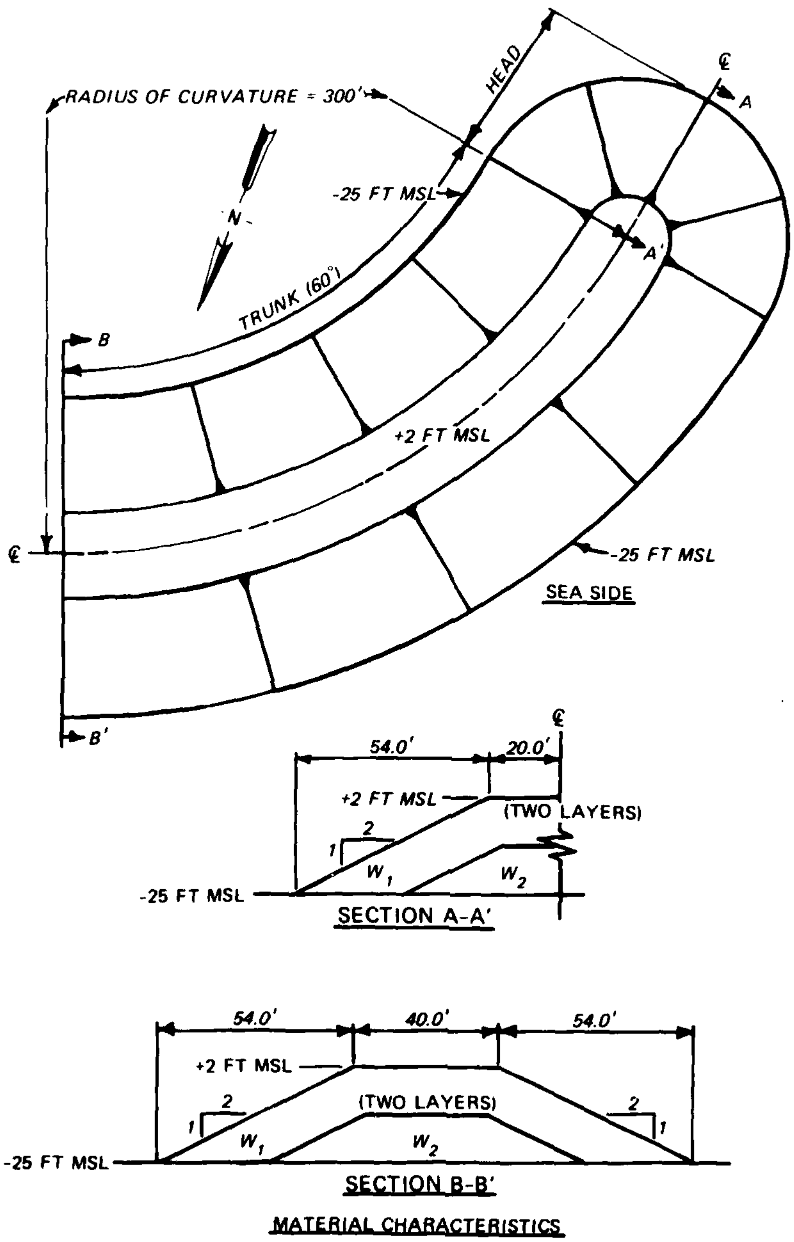
WEST SLOPE
PLAN W-3



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.031-LB ROCK @ 165 PCF	W_1 = 2,000-LB ROCK @ 165 PCF
W_2 = 0.003-LB ROCK @ 165 PCF	W_2 = 200-LB ROCK @ 165 PCF

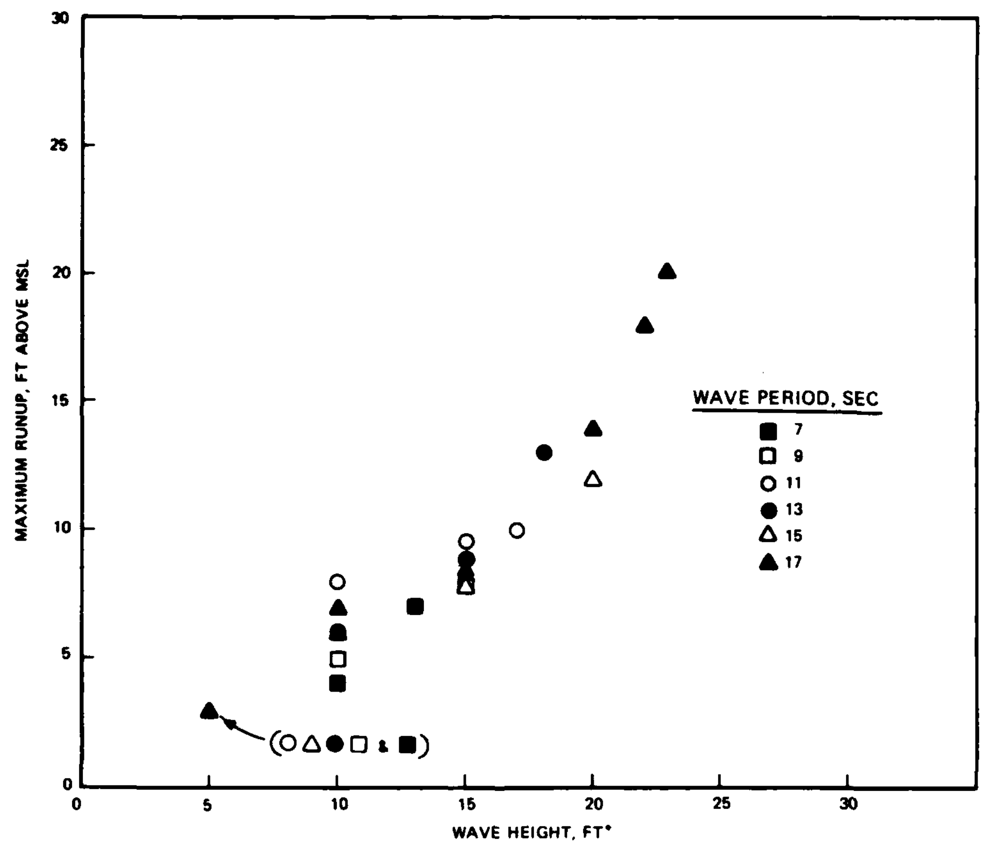
WEST SLOPE
PLAN W-4



MATERIAL CHARACTERISTICS

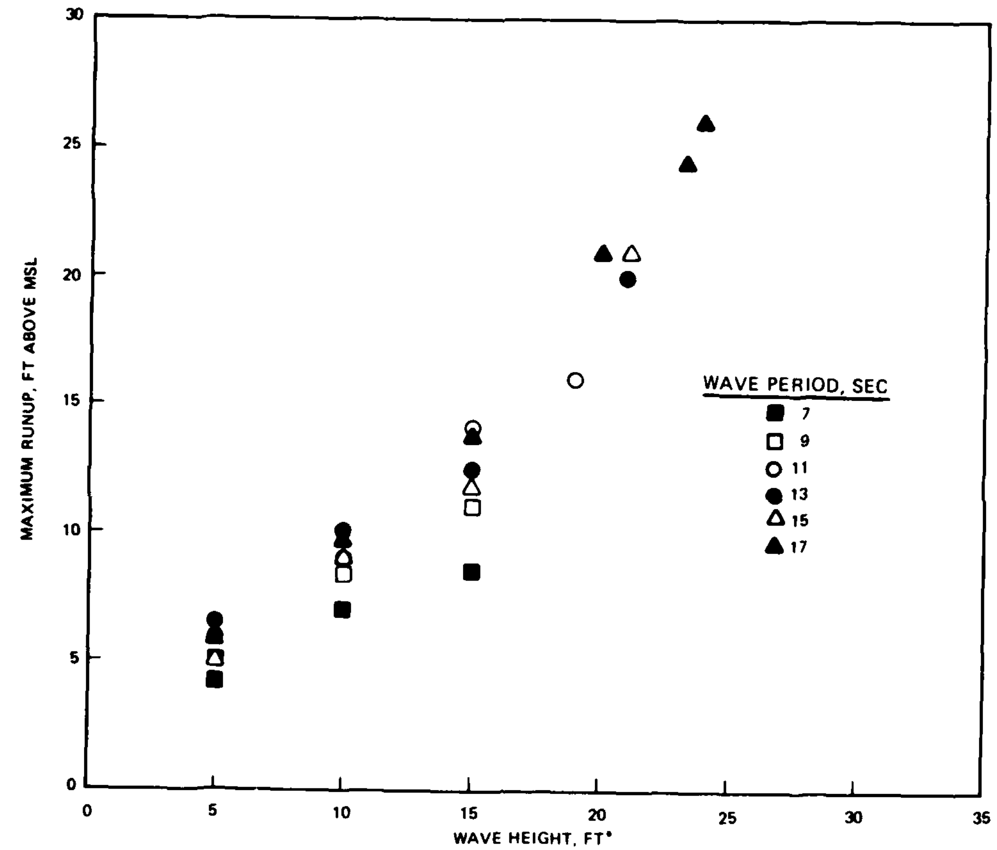
MODEL	PROTOTYPE
W_1 = 0.380-LB ROCK @ 165 PCF	W_1 = 55,350-LB ROCK @ 165 PCF
W_2 = 0.019-LB ROCK @ 165 PCF	W_2 = 2,768-LB ROCK @ 165 PCF

OFFSHORE BREAKWATER
PLAN 3D-1



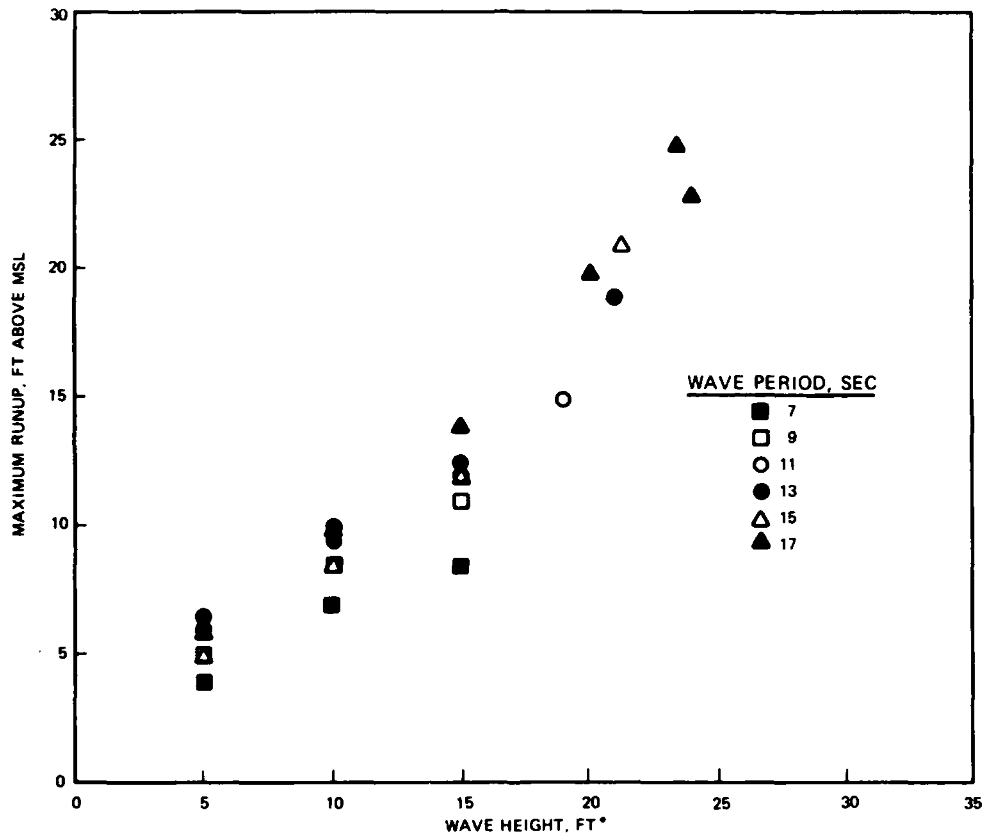
* INCIDENT WAVE HEIGHT MEASURED DURING CALIBRATION AT THE -21.0 FT MSL CONTOUR.

MAXIMUM RUNUP, NORTH SLOPE
PLAN N-2, ROUGH SLOPE
SWL = 0.0 FT MSL



* INCIDENT WAVE HEIGHT MEASURED DURING CALIBRATION AT THE -21.0 FT MSL CONTOUR.

MAXIMUM RUNUP, NORTH SLOPE
PLAN N-2, ROUGH SLOPE
SWL = + 1.9 FT MSL

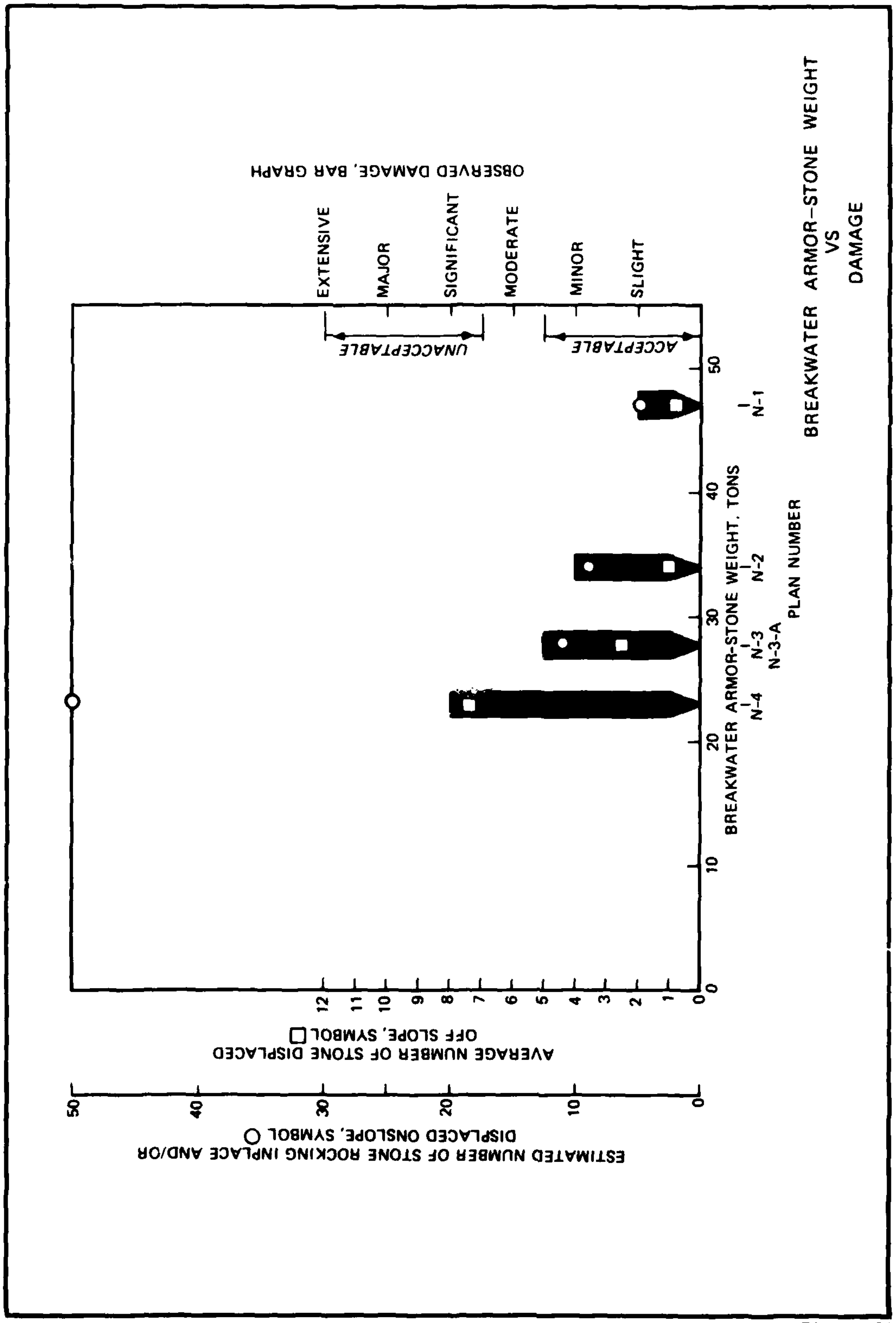


* INCIDENT WAVE HEIGHT MEASURED DURING CALIBRATION AT THE -21.0 FT MSL CONTOUR.

MAXIMUM RUNUP, NORTH SLOPE

PLAN N-3-A, ROUGH SLOPE

SWL = +1.9 FT MSL



BREAKWATER ARMOR-STONE WEIGHT
VS
DAMAGE
OBSERVED DAMAGE, BAR GRAPH
EXTENSIVE
MAJOR
SIGNIFICANT
MODERATE
MINOR
SLIGHT
UNACCEPTABLE
ACCEPTABLE
AVERAGE NUMBER OF STONE DISPLACED OFF SLOPE, SYMBOL □
ESTIMATED NUMBER OF STONE ROCKING INPLACE AND/OR DISPLACED ONSLOPE, SYMBOL ○
0
1
2
3
4
5
6
7
8
9
10
11
12
0
10
20
30
40
50
BREAKWATER ARMOR-STONE WEIGHT, TONS
0
10
20
30
40
50
N-1
N-2
N-3
N-3-A
N-4
PLAN NUMBER

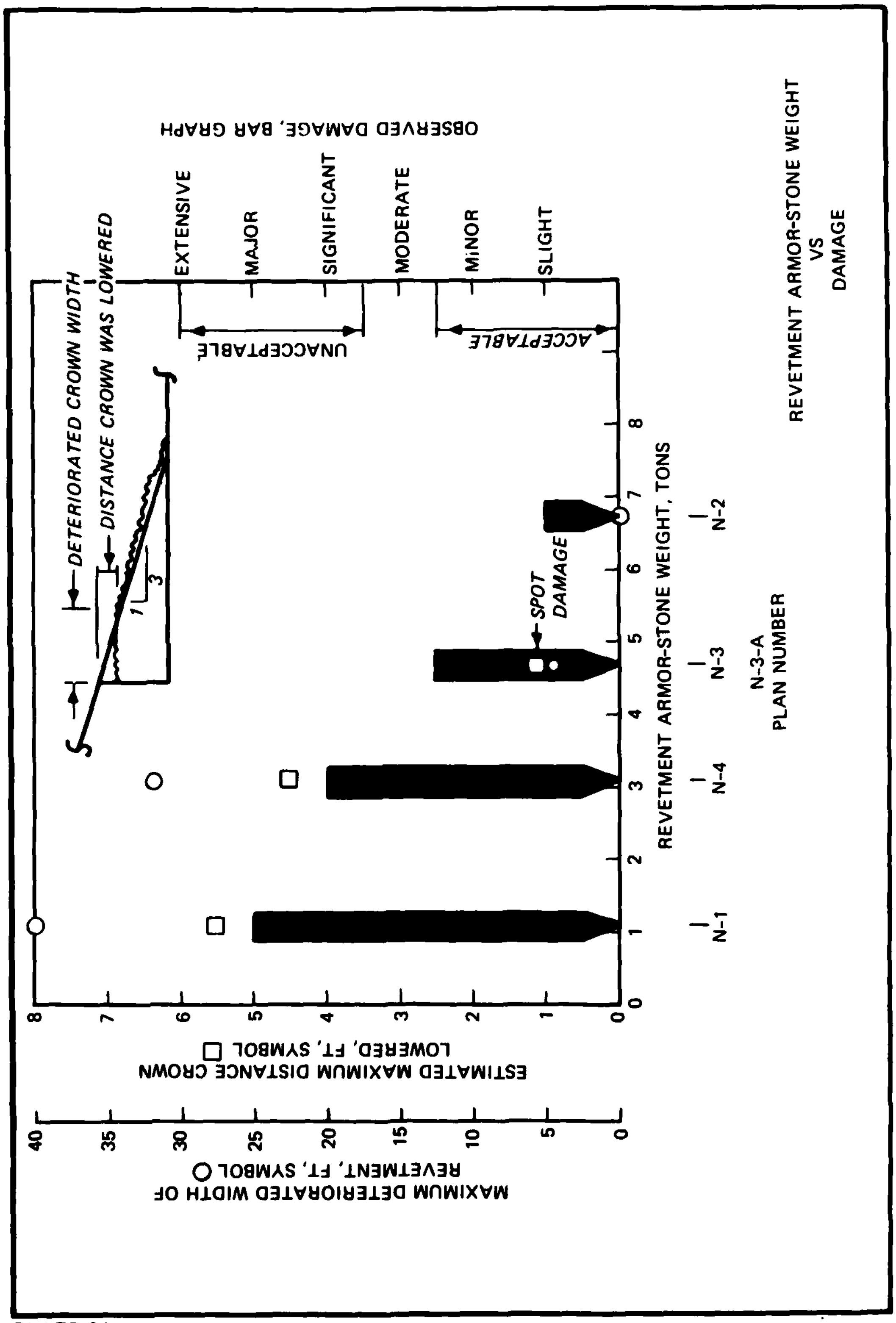
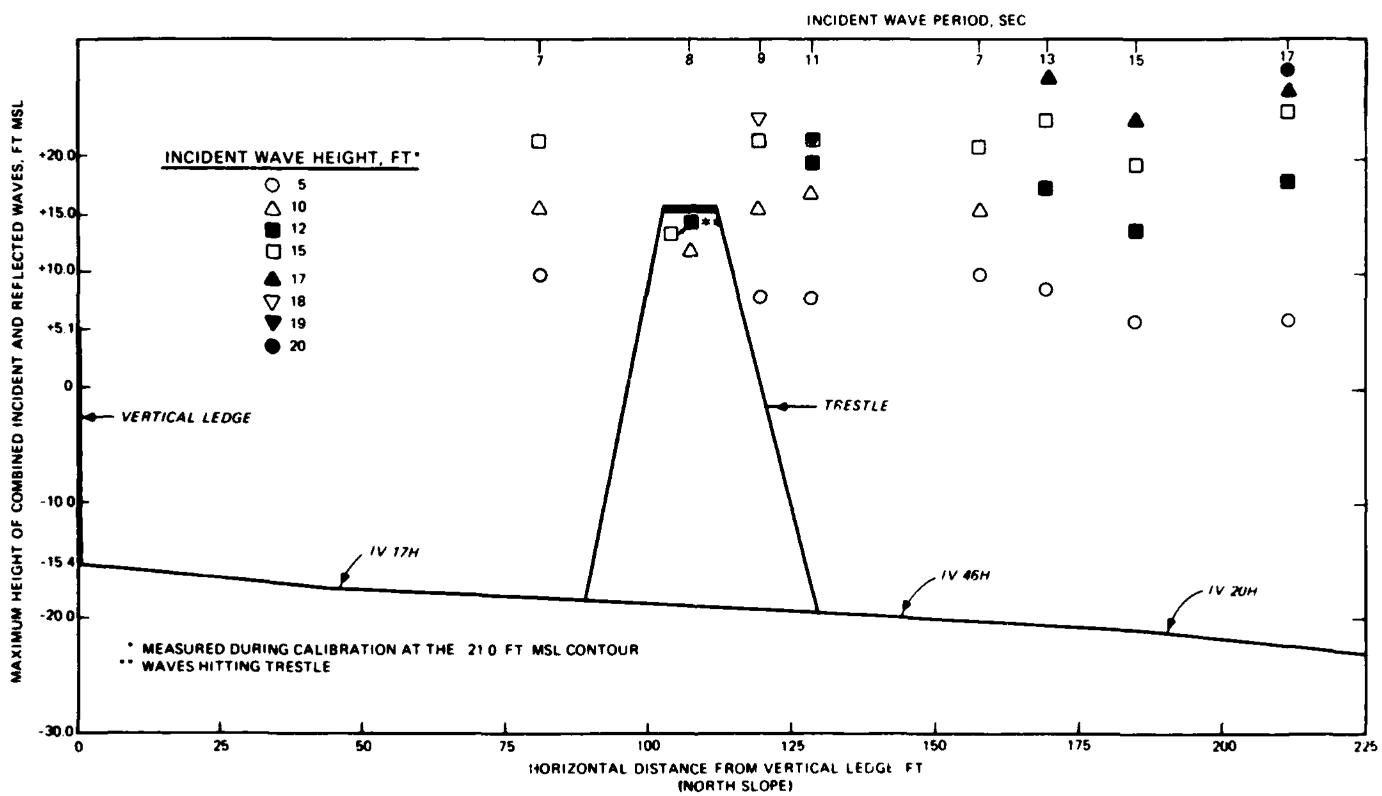
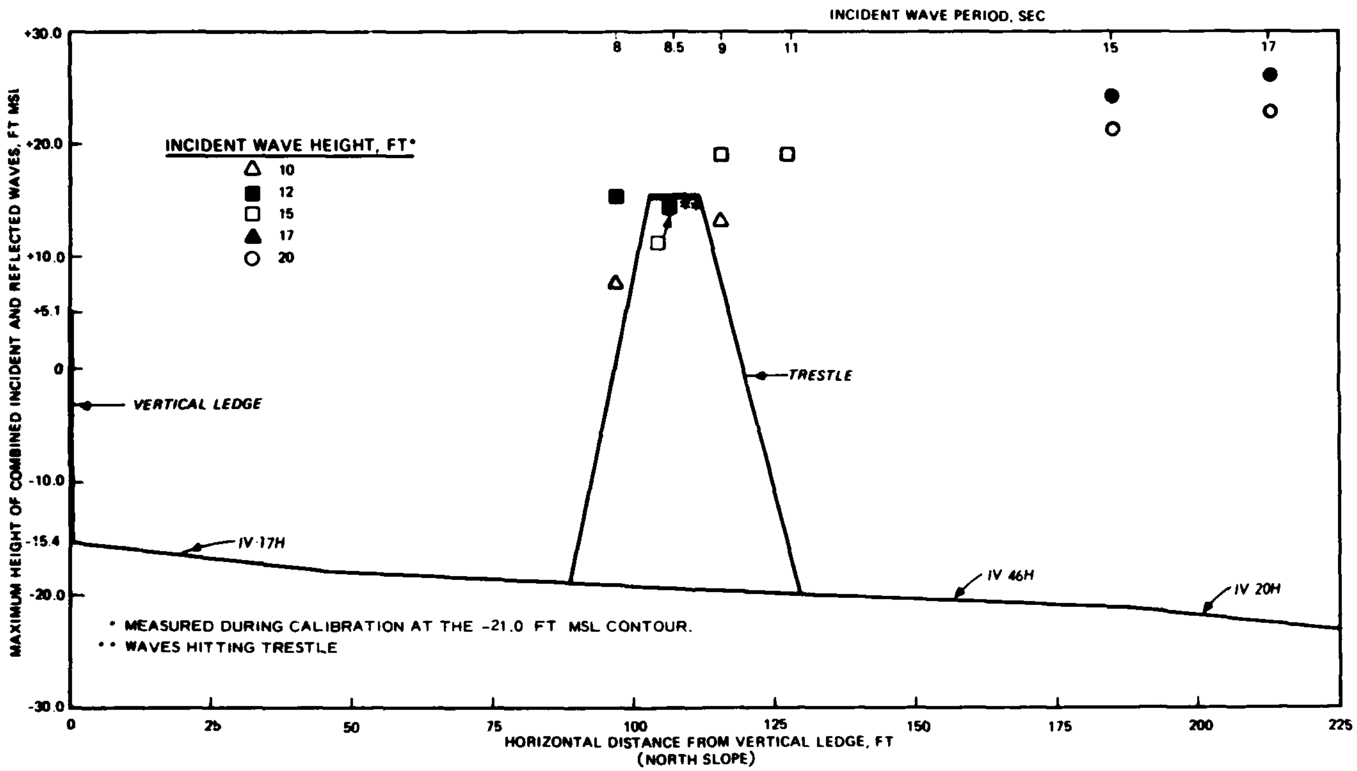

REVETMENT ARMOR-STONE WEIGHT
VS
DAMAGE
OBSERVED DAMAGE, BAR GRAPH
EXTENSIVE
MAJOR
SIGNIFICANT
MODERATE
MINOR
SLIGHT
UNACCEPTABLE
ACCEPTABLE
DETERIORATED CROWN WIDTH
DISTANCE CROWN WAS LOWERED
1
3
SPOT DAMAGE
ESTIMATED MAXIMUM DISTANCE CROWN LOWERED, FT, SYMBOL □
MAXIMUM DETERIORATED WIDTH OF REVETMENT, FT, SYMBOL ○
REVETMENT ARMOR-STONE WEIGHT, TONS
0
1
2
3
4
5
6
7
8
0
5
10
15
20
25
30
35
40
N-1
N-4
N-3
N-2
N-3-A
PLAN NUMBER

PLATE 24



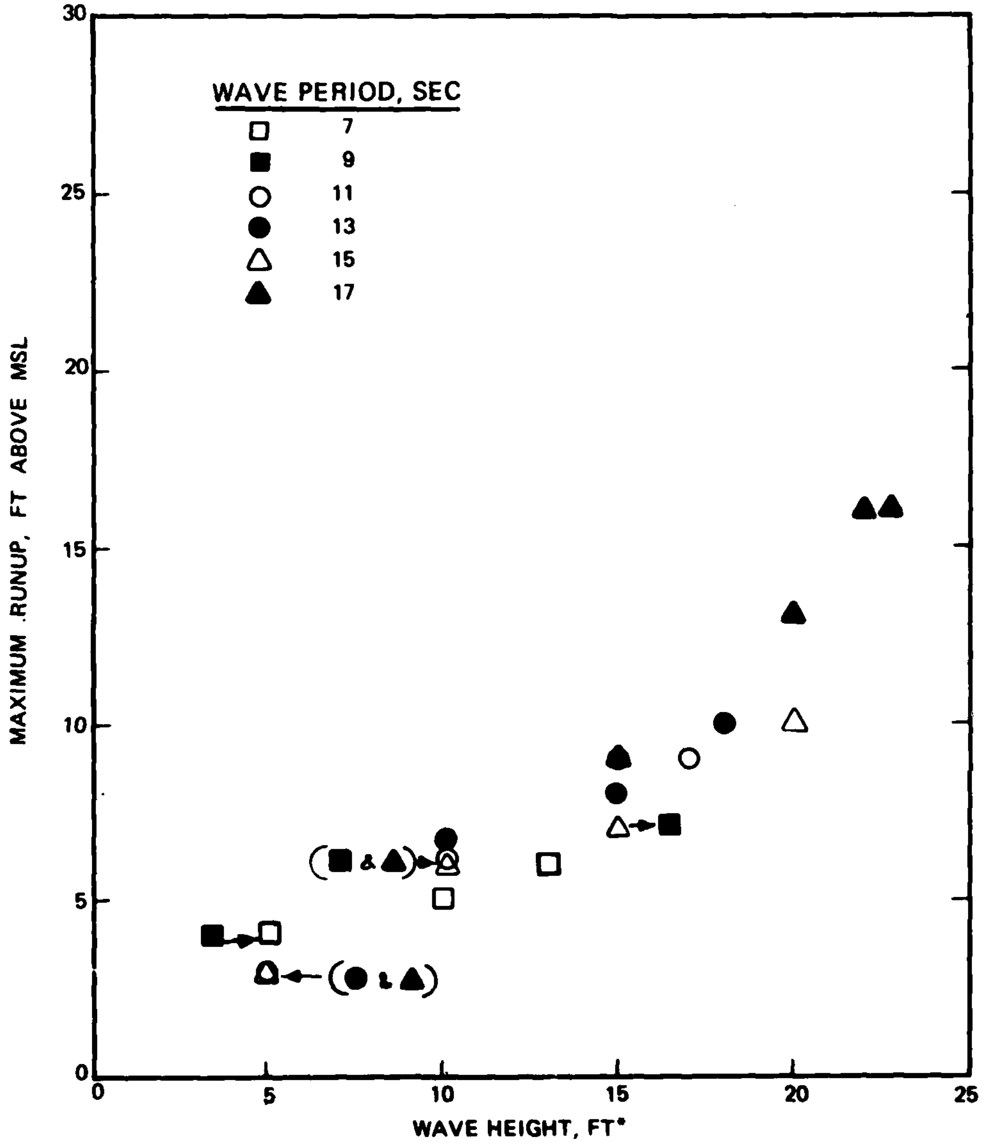
LOCATION AND MAXIMUM HEIGHT OF COMBINED INCIDENT AND REFLECTED WAVES VS INCIDENT WAVE PERIOD

SWL = +1.9 FT MSL



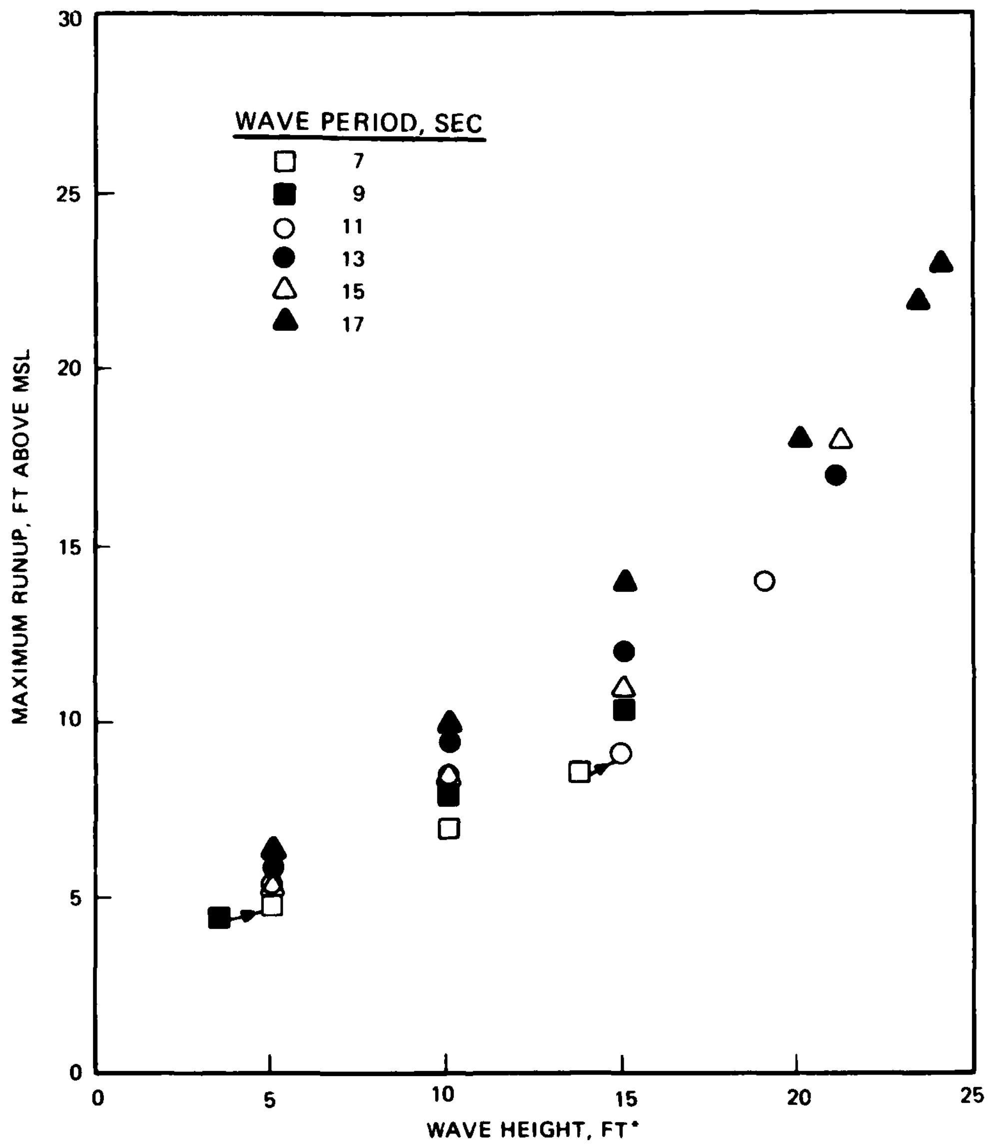
LOCATION AND MAXIMUM HEIGHT OF
COMBINED INCIDENT AND REFLECTED WAVES
VS
INCIDENT WAVE PERIOD
SWL = 0.0 FT MSL

PLATE 26



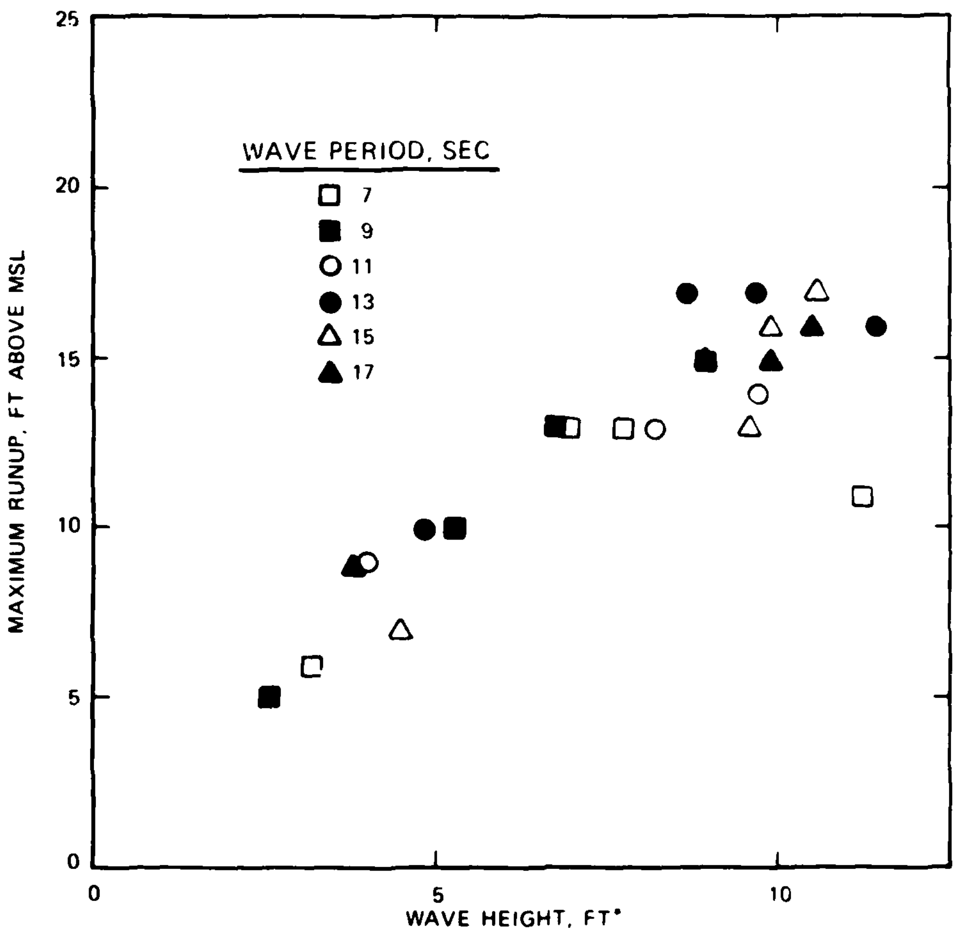
* INCIDENT WAVE HEIGHT MEASURED DURING CALIBRATION AT THE -21.0 FT MSL CONTOUR.

MAXIMUM RUNUP, NORTH SLOPE
PLAN N-6, ROUGH SLOPE
SWL = 0.0 FT MSL
CULVERT OPEN



* INCIDENT WAVE HEIGHT MEASURED DURING CALIBRATION AT THE -21.0 FT MSL CONTOUR.

MAXIMUM RUNUP, NORTH SLOPE
PLAN N-6, ROUGH SLOPE
SWL = +1.9 FT MSL
CULVERT OPEN



* WAVE HEIGHTS MEASURED DURING CALIBRATION AT THE - 13.0 FT MSL ELEVATION.

MAXIMUM RUNUP, WEST SLOPE
PLAN W-2, SWL = +1.9 FT MSL

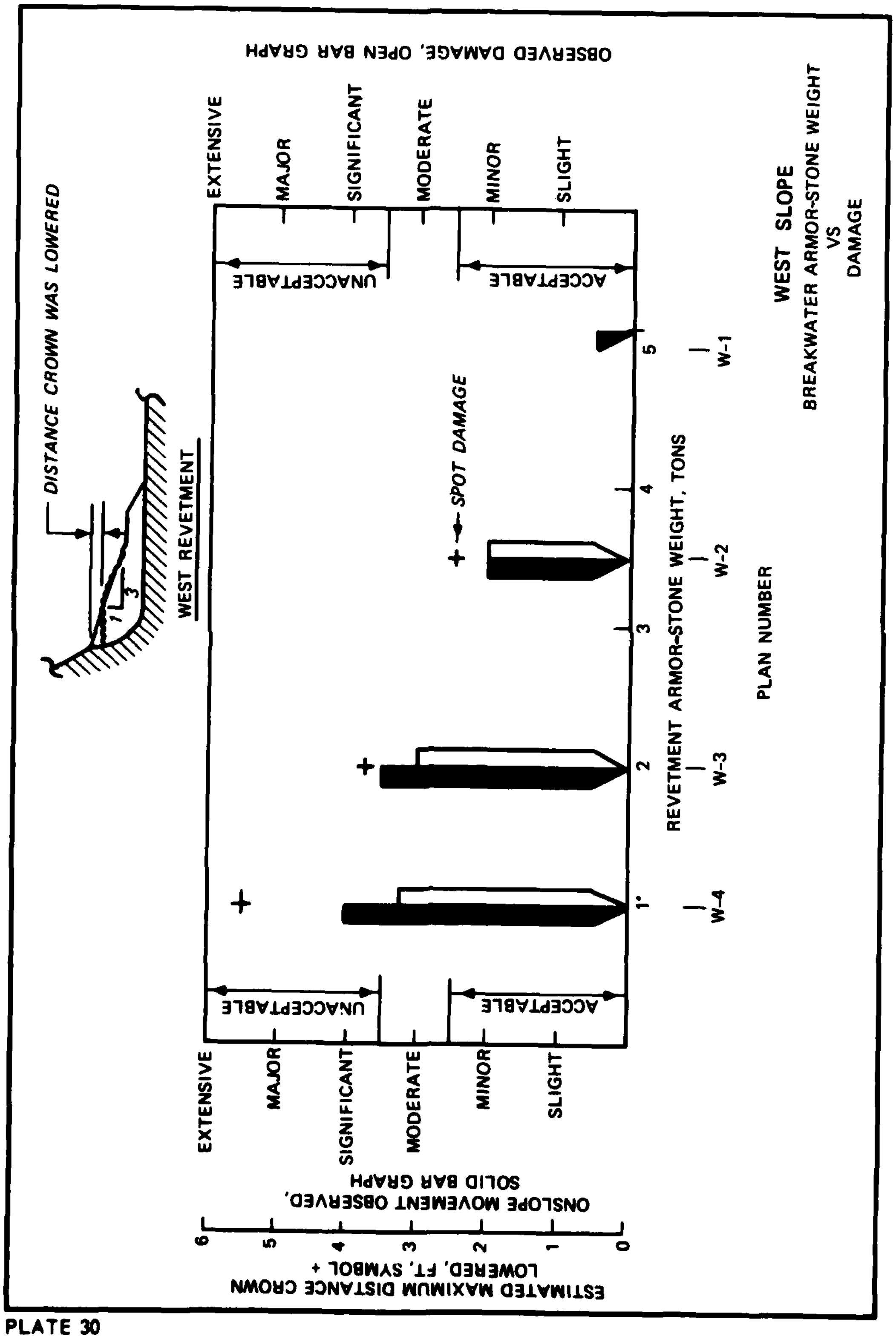
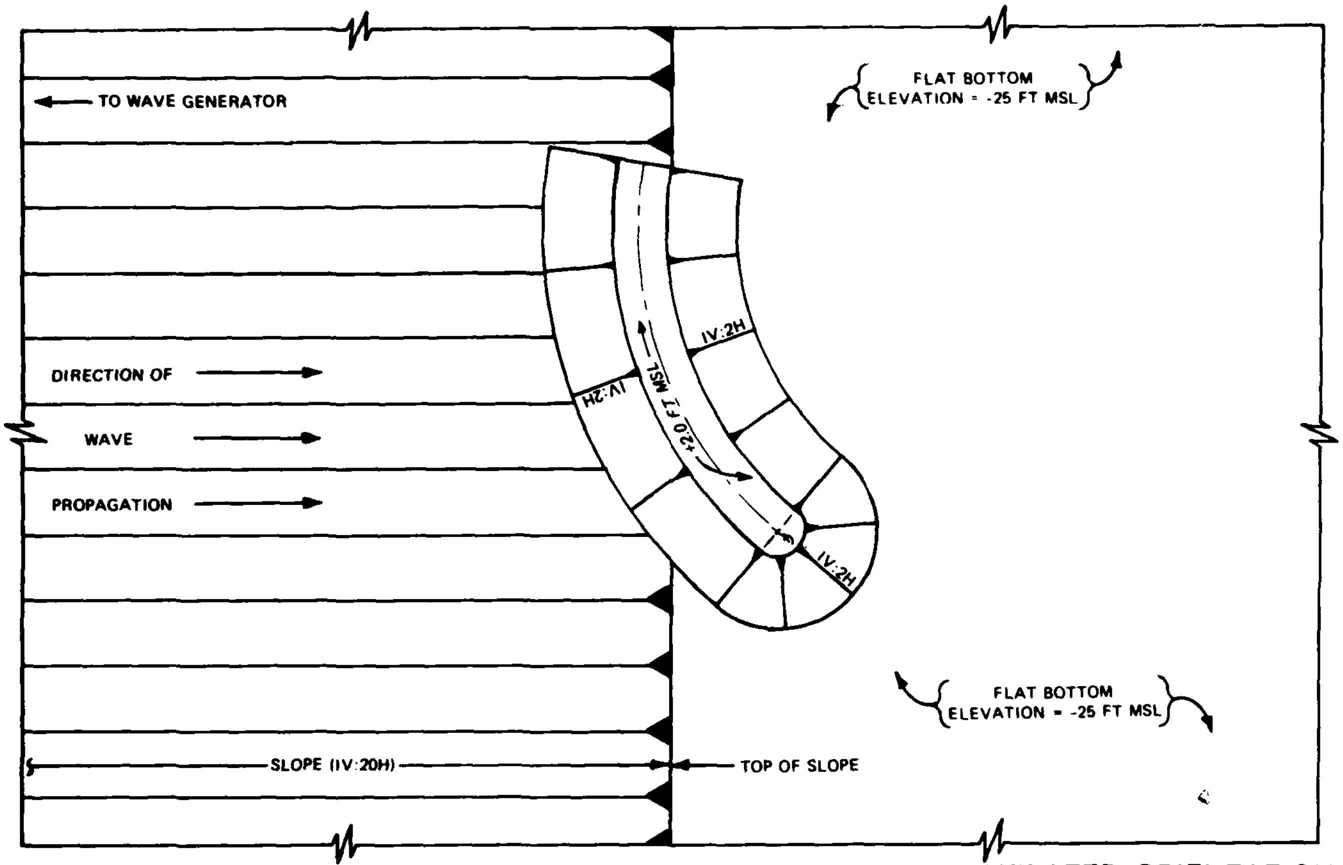
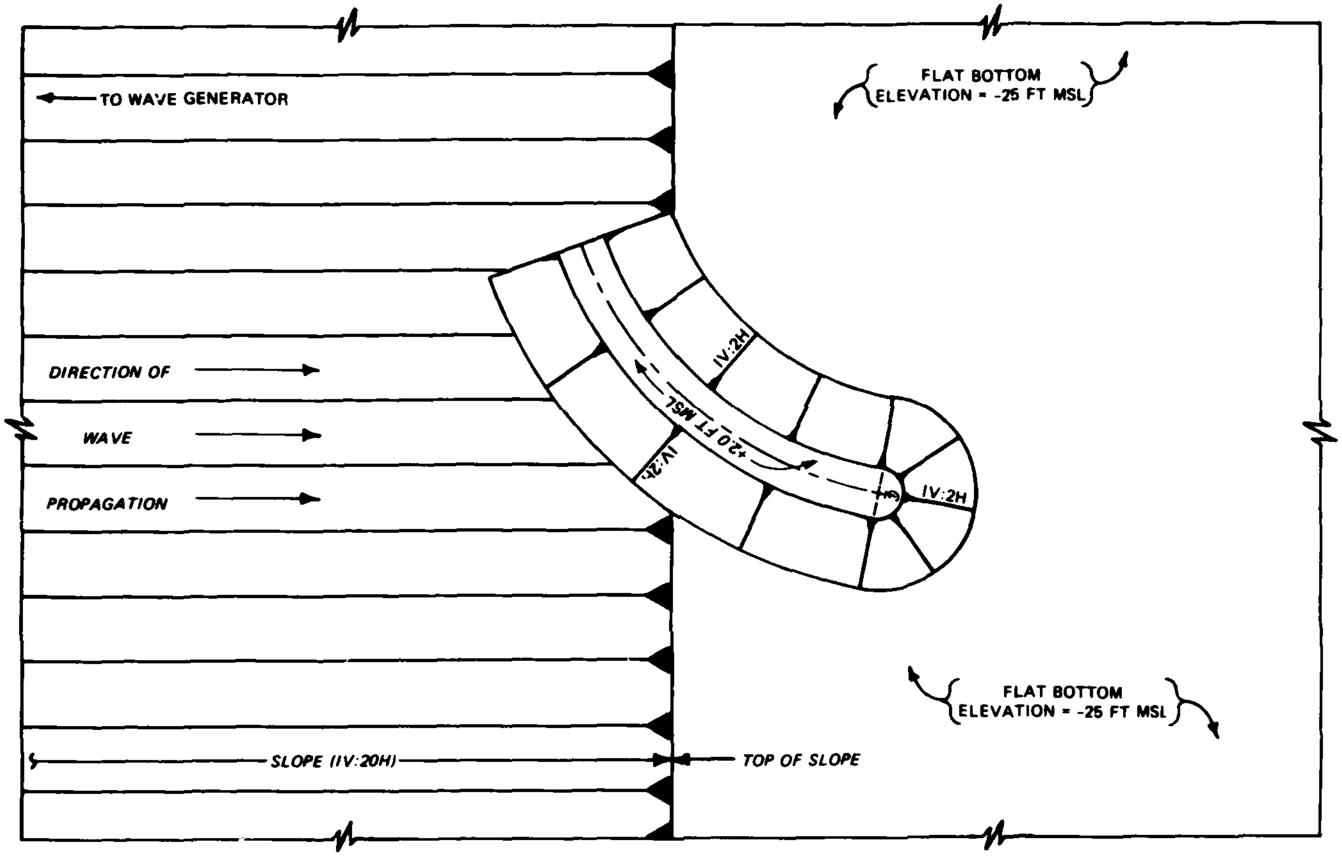

WEST SLOPE
BREAKWATER ARMOR-STONE WEIGHT
VS
DAMAGE
DISTANCE CROWN WAS LOWERED
WEST REVETMENT
1
3
OBSERVED DAMAGE, OPEN BAR GRAPH
EXTENSIVE
MAJOR
SIGNIFICANT
MODERATE
MINOR
SLIGHT
UNACCEPTABLE
ACCEPTABLE
SPOT DAMAGE
1*
2
3
4
5
W-4
W-3
W-2
W-1
REVETMENT ARMOR-STONE WEIGHT, TONS
PLAN NUMBER
ONSLOPE MOVEMENT OBSERVED,
SOLID BAR GRAPH
ESTIMATED MAXIMUM DISTANCE CROWN
LOWERED, FT, SYMBOL +
0
1
2
3
4
5
6

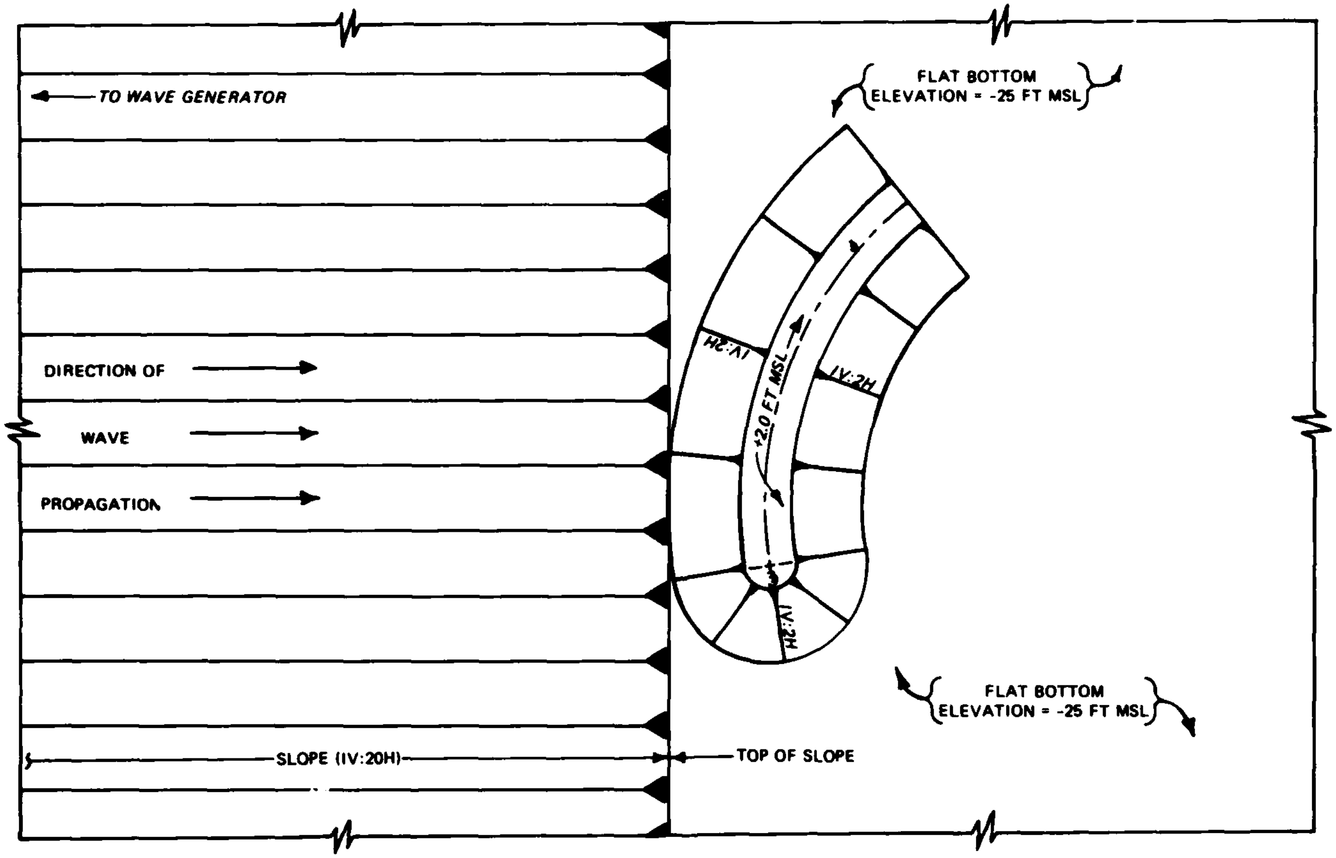
PLATE 30



BREAKWATER ORIENTATION
3-D TEST FLUME FOR WAVE ATTACK
FROM NORTH 30° WEST



BREAKWATER ORIENTATION IN
3-D TEST FLUME FOR WAVE ATTACK
FROM THE NORTH



BREAKWATER ORIENTATION IN
3-D TEST FLUME FOR WAVE ATTACK
FROM NORTH 72° WEST

APPENDIX A: NOTATION

A	Area, ft^2
H	Wave height, ft
L	Length, linear scale, ft
msl	Mean sea level
swl	Still-water level
S	Specific gravity
T	Time
W	Weight, lb
α	Angle of breakwater slope, measured from horizontal, deg
γ	Specific weight, pcf

Superscripts

a	Refers to ratio of model quantities to prototype quantities (i.e., a = m/p)
m	Refers to model quantities
p	Refers to prototype quantities
r	Refers to armor stone
w	Refers to water in which the structure is situated
1-4	Refers to different stone sizes

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Markle, Dennis G.
Breakwater and revetment stability study San Juan National Historic Site, San Juan, Puerto Rico : hydraulic model investigation / by Dennis G. Markle (Hydraulics Laboratory, U.S. Army Engineer Waterways Experiment Station). -- Vicksburg, Miss. : The Station ; Springfield, Va. ; available from NTIS, 1981.
195 p. in various pagings , 33 p. of plates : ill. ; 27 cm. -- (Technical report / U.S. Army Engineer Waterways Experiment Station ; HL-81-11)
Cover title.
"September 1981."
Final report.
"Prepared for U.S. Army Engineer District, Jacksonville and The National Park Service, Southeast Regional Office, U.S. Department of the Interior."

1. Breakwaters. 2. Erosion. 3. Hydraulic models. 4. San Juan National Historic Site (Puerto Rico). 5. Shore protection. I. United States. Army. Corps of

Markle, Dennis G.
Breakwater and revetment stability study San Juan : ... 1981.
(Card 2)

Engineers. Jacksonville District. II. United States. National Park Service. III. U.S. Army Engineer Waterways Experiment Station. Hydraulics Laboratory. IV. Title V. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; HL-81-11.
TA7.W34 no.HL-81-11

DATE
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